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State Railroad Taxation

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STATE RAILROAD TAXATION

The amount of taxes paid by railroads of the United States during the fiscal year of 1903 was, according to the reports of the interstate commerce commission, $57,797,737, an increase over the amounts paid in 1902 of $11,089,902, and it seems, that no matter in what light we view the assessment and collection of these taxes, the problem is a fundamental one.

A survey of the methods of assessment, levy, and collection of these taxes as practiced in the different states reveals the fact of confusion, and uncertainty, not only with the agencies of government, in trying to overcome the liability and incentive to tax evasion, but to the railroads themselves because of being liable to double taxation in many instances.

Not only is a line of railroad crossing state boundaries liable to taxation from two different and possibly entirely unsympathetic systems, but the minor civil divisions and municipalities of one commonwealth, under our present system, exercise an independence in administration which occasions yet further confusion and uncertainty.

Railroad property was not originally taxed differently from other kinds of property. Its peculiarities, its methods of capitalzaition and its significances from a social or industrial point of view were not early recognized, or if so, were not deemed of importance sufficient to warrant different treatment from that of general property. The State of Pennsylvania perhaps presenting the only exception in having differentiated its treatment of railroads from the start.

The true history of railway taxation begins with the application of the general property tax to the railway corporations, which tax was administered by the local officials. The method of taxation of general property, according to the value thereof, as assessed by local officials, being in force at the time railways were first constructed, was consequently applied to railways and although inadequate to reach the real taxable capacity of these corporations the practice is still in vogue in many states. Rhode Island is a conspicuous example of the operation of the general tax system, for in that state there is no distinction between railroad corporations, or in fact any corporation, and the individual for purposes of taxation. The same, in general, was true of Louisiana at the beginning of the decade, although unlike Rhode Island, there were special statutes upon the subject. The effect of these statutes, however, was not such as to render the taxation of corporations different from the taxation of other properties.

The principles of local self government have become so firmly fixed that they are a real hindrance to a proper adjustment of the laws of taxation. The paramount thought running through these principles of local self government is, that as all persons and property within the jurisdiction of a town or county must come under the protection of their authorities, so must the corresponding rights of taxation of such persons or property be controlled by these authorities in order to be able to afford such protection. The power to locally tax any article or species of property once acquired, is reluctantly given up and there are many advocates of the right of local taxation applied to every species of property without regard to its possible complex or interstate character. In fact this local self government is so strong in some of the states that there still exists the local taxation of some parts of railroad property actually used in their operation as common carriers.

The opposition to giving up their local sources of revenue has been so strong that it has blocked the attempts to secure methods of taxation along more scientific lines and has prevented the severing of corporation taxation from the general property tax on real estate and tangible personality. The strong disinclination of legislators to antagonize local interests and to do anything which would tend to disrupt the established order of things has led them, in many cases, to permit local taxing districts to maintain their hold on property which they have become accustomed to tax, and to cover the whole field by laws of special enactment, taxing franchises, gross earnings, stocks and bonds, etc., and aiming in this manner to impose taxes equivalent to the burden which the railroads ought in justice to bear.

Instead of tearing the whole structure of the old method out by its roots and establishing a new and comprehensive system they have, by a process of grafting and patchwork, produced an exhibit on the whole confused and chaotic and one which is alike unsatisfactory to both the agencies of government and the railroads themselves.

There is, however, a seeming uniformity of method among the different states in subjecting all real estate, not connected with the railroad track, or necessary to the railway operations as common carriers, to local assessment in the taxation district where located, the same as the property of individuals.

The total inadequacy of the general property tax, as locally assessed, in reaching the full taxable value of corporate bodies, when applied to them, seems to have become gradually recognized and as other elements in corporate organization presenting additional value, appeared, they were eagerly seized upon for taxation purposes to make up this deficiency.

The most important of these elements was the franchise, and here we are brought face to face with the question, What is a franchise? Blackstone defines it, in terms rather vague for our purpose, as "A royal privilege or branch of the King's prerogative subsisting in the hands of a subject." The United States Supreme Court has handed down this definition: "A franchise is a right, privilege or power of public concern which ought not to be exercised by private individuals at their mere will and pleasure; but which should be reserved for public control and administration, either by the government directly or by public agents acting under such conditions and regulations as the government may impose in the public interest and for the public security."

The general use of the term franchise seems to indicate that this definition is rather narrow as there is a common tendency to grant the franchise privilege to strictly private corporations rather more often than to the interests of public control. The franchise then, we conclude, is the right to be a corporation, to acquire and possess property, to take the lands of private citizens by the sovereign right of emminent domain for corporation use upon the payment of just compensation therefor, and to fix and collect charges for the transportation of traffic. Under this definition the franchise tax would become a tax on the privilege of doing business, and not a tax on property, and it has been so held and sustained by the courts, thereby avoiding the constitutional provisions, as found in nearly every state, requiring uniformity or equal and proportionate taxation of property. The origin of the franchise tax is plainly traceable to the insufficiency and evils of the general property tax as applied to corporations.

The definitions of a franchise, of what it really consists, and how its values shall be measured and set out are points of variance between the different states and a presentation of the questions relating to the taxation of franchises is outside the scope of this paper. It is sufficient for the purpose that there is no common standard of valuation acceptable to the different states. The laws governing the valuation, while often of the same general tone and scope, have received different judicial interpretations and construction so that state statutes, even in the same language, have often wide difference in important particulars of their administration.

While a detailed description of the laws of State Railroad taxation is entirely outside the field of this paper, attention is respectfully called to the concise description of the taxation laws of each state and territory arranged and published by the Interstate Commerce Commission under the head of "Railways in the United States in 1902, Part V." This report is exhaustive and is arranged for each state on a uniform plan so that any particular fact for any state may be easily ascertained, side references to the constitution and codes being given in each case.

In order to give the clearest idea of the amounts and nature of the taxes paid by the railroads of the United States, the folloing extracts and tables are taken from the "Statistics of Railways in the United States in 1903."

TAXES.

"The interest which centers in the general question of railway taxation, both from the corporate and the public point of view, warrants the continuance of the table which shows the amount of taxes collected from railways in each state. The amount per mile of line, as well as the aggregate amount, is given in each case.

SUMMARY SHOWING TAXES AND ASSESSMENTS OF THE RAILWAYS IN THE UNITED STATES, BY STATES AND TERRITORIES, FOR THE YEAR ENDING JUNE 30, 1903.

State or Territory	Amount	Per Mile of Line
Alabama	$ 783,389	$186
Arkansas	546,996	167
California	1,588,860	281
Colorado	1,357,464	285
Connecticut	1,050,222	1,025
Delaware	99,319	296
Florida	472,132	148
Georgia	700,601	122
Idaho	328,884	227
Illinois	4,656,633	410
Indiana	2,947,897	437
Iowa	1,719,654	182

State or Territory	Amount	Per Mile of Line
Kansas	2,197,136	251
Kentucky	1,180,290	379
Louisiana	709,466	257
Maine	404,191	208
Maryland	467,669	353
Massachusetts	2,957,581	1,413
Michigan	2,398,437	296
Minnesota	2,144,872	296
Mississippi	538,138	181
Missouri	1,474,185	206
Montana	667,453	208
Nebraska	1,152,323	199
Nevada	217,246	231
New Hampshire	402,864	327
New Jersey	1,684,077	768
New York	4,945,811	604
North Carolina	476,438	128
North Dakota	640,482	210
Ohio	3,713,640	426
Oregon	274,161	171
Pennsylvania	4,572,166	447
Rhode Island	189,435	894
South Carolina	455,906	150
South Dakota	298,093	100
Tennessee	770,218	245
Texas	1,179,190	108
Utah	372,296	235
Vermont	149,107	142
Virginia	808,280	216
Washington	747,556	248
West Virginia	520,557	227
Wisconsin	1,887,375	284
Wyoming	174,973	141
Arizona	197,437	125
District of Columbia	31,257	1,003
Indian Territory	15,712	8
New Mexico	258,674	118
Oklahoma	270,994	146
Total	(1)57,797,737	290

(1) Excludes $51,832 paid in the Dominion of Canada; $9.157 internal revenue not localized by states and territories; $409 "On the value of real and personal property," paid directly to the United States government, and $5,156 not distributed by states.

Certain remarkable changes are observed, if one compares the amounts paid per mile of line in the same States for the two years in question. The following States and Territories show a falling off in the rate of taxation per mile of line: Alabama, Arkansas, Delaware, Idaho, Illinois, Indiana, Nebraska, New Jersey, New York, North Carolina, South Carolina, Texas, Vermont, Indian Territory, and New Mexico. All the other States and Territories with the exception of Kansas, North Dakota and Wyoming, where the payment is the same as for the previous year, show an increase in the tax payment per mile of line. The more important increases are the following: District of Columbia, $446; Michigan, $106; Kentucky, $83; Montana, $79; Ohio, $72; Maryland, $69; Mississippi, $51; Minnesota, $49; Nevada, $38; California, $34; Washington, $33; Arizona, $33; Utah, $29; Wisconsin, $25; and other States a less amount. The increased payment for taxes per mile of line in the entire country was $16, being $274 in the fiscal year 1902 and $290 in 1903.

The next summary presents a classification of railway taxes by States and Territories. Especial care has been taken to secure a uniform interpretation of the kinds of taxes designated, and the information submitted throws considerable light upon this difficult subject.

ANALYSIS OF TAXES BY STATES AND TERRITORIES, SHOWING THE BASIS OF PAYMENTS ACCORDING TO THE VARIOUS LAWS UNDER WHICH RAILWAYS ARE TAXED.

STATE OR TERRITORY	AD VALOREM TAX		SPECIFIC TAX		
	On the Value of Real and Personal Property	On the Value of Stocks or Bonds or on Valuation Based on Earnings, Dividends or Other Results of Operation	On Stocks, Bonds, Loans, Etc.	On Gross or Net Earnings, Revenue or Dividends	On Traffic or Some Physical Quality of Property Operated or on Privilege
Alabama	$ 756,662	$	$	$ 14,871	$ 5,604
Arkansas	546,996				
California	1,486,358	80			675
Colorado	1,351,197		375		
Connecticut	33,946	1,016,276			
Delaware	18,754	500			78,326
Florida	472,085				
Georgia	677,146			1,523	
Idaho	327,846				
Illinois	3,572,218			1,026,651	
Indiana	2,907,633				
Iowa	1,717,049				
Kansas	2,196,900				
Kentucky	845,751	333,356			
Louisiana	697,404				10,917
Maine (1)	60,968			328,348	
Maryland	123,771		3,960	311,637	258
Massachusetts	1,153,358	1,658,385		21,593	
Michigan	698,264			1,676,810	
Minnesota	32,348			2,102,439	
Mississippi	467,225				70,539
Missouri	1,471,871				1,000
Montana	667,453				
Nebraska	1,149,861				
Nevada	206,589				
New Hampshire	390,047				
New Jersey	1,168,369	449,457			3,690
New York (2)	4,168,369	404,634	18,400	259,898	5,095
North Carolina	457,308				16,318
North Dakota	640,482				
Ohio (3)	2,578,028		100	1,008,045	98
Oregon	266,529				405
Pennsylvania (4)	690,298		382,164	697,392	190
Rhode Island	188,116				
South Carolina	445,931				2,520
South Dakota	298,093				
Tennessee	758,347				11,871
Texas (5)	1,083,062		2,270	92,092	115
Utah	368,470				
Vermont	136		849	142,242	
Virginia	765,905			35,841	1,376
Washington	745,830				
West Virginia	495,388		12,249		21
Wisconsin	27,438			1,836,065	1,867
Wyoming	174,898				
Arizona	196,221				
District of Col	27,428			903	
Indian Territory	174,898				73
New Mexico	196,221				
Oklahoma	270,994				
Total (6)	$40,120,149	$6,397,331	$420,367	$9,559,350	$210,958

(1) Excludes $121, unclassified. (2) Excludes $2,479, unclassified. (3) Excludes $702, unclassified. (4) Excludes $745, unclassified. (5) Excludes $539, unclassified. (6) Excludes $56.418, as follows: $37,110 paid in Dominion of Canada; $9,157 internal revenue, not localized by States and Territories; $409, "On the value of real and personal property," paid directly to the United States Government $5,156, not distributed by States; and $4,586, unclassified.

ANALYSIS OF TAXES, BY STATES AND TERRITORIES, SHOWING THE BASIS OF PAYMENTS ACCORDING TO THE VARIOUS LAWS UNDER WHICH RAILWAYS ARE TAXED. (Cont'd).

STATE OR TERRITORY	On Property Owned, Not Used in Operation, and Miscellaneous	Internal Revenue, United States Government
Alabama	$ 6,256	
Arkansas		
California	100,474	1,273
Colorado	5,892	
Connecticut		
Delaware	1,739	
Florida	47	
Georgia	21,932	
Idaho	1,038	
Illinois	57,428	336
Indiana	40,197	
Iowa	2,605	
Kansas	236	
Kentucky	1,182	
Louisiana	1,145	
Maine	14,754	
Maryland	28,031	12
Massachusetts	124,245	
Michigan	23,262	101
Minnesota	10,061	24
Mississippi	374	
Missouri	1,310	
Montana		
Nebraska	2,462	
Nevada	10,657	
New Hampshire	12,817	
New Jersey	61,241	1,020
New York	84,610	2,326
North Carolina	2,050	762
North Dakota		
Ohio	126,628	39
Oregon	7,227	
Pennsylvania	264,964	1,770
Rhode Island	1,319	
South Carolina	6,930	525
South Dakota		
Tennessee		
Texas	107	1,005
Utah	3,826	
Vermont	2,852	28
Virginia	5,158	
Washington	1,726	
West Virginia	12,899	
Wisconsin	22,005	
Wyoming	75	
Arizona	1,211	5
Dist. of Col.	2,387	
Indian Ter.		
New Mexico		
Oklahoma	343	4
Total	$1,075,699	9,297

The appearance of small amounts in certain columns of the above summary may indicate a failure on the part of some of the reporting carriers to interpret the laws under which they pay taxes in conformity with the principles of classification prescribed, but in the majority of cases they indicate some peculiarity in the conditions of the charter, either of the carrier or of the municipality in which the property of the carrier lies. In making use of the above summary, these small amounts may safely be considered as representing exceptions to the established rules of taxation in the states where they occur.

The methods and combinations of methods of taxing operated railroad property, now in force in the several states, are strikingly exhibited in the last table. The general classification of ad valorem taxation of real and personal property in the first column and upon the value of stocks or bonds earnings and other results of operation in the second column, does not disclose the various provisions of the laws which produce the results of the table.

The ad valorem tax or tax on the value of the whole or some part of the real and personal property of the railroads prevails in every state and territory, including the District of Columbia, with the exception of Vermont.

The specific tax on gross or net earnings, revenue or dividends, in connection with the property tax exists in fifteen states and the District of Columbia.

The specific tax on earnings is the exclusive tax on property employed in Railroad operation in the states of Illinois, Maine, Massachusetts, Michigan, Minnesota, Ohio, Vermont, and constitutes the largest parts of Wisconsin, Maryland and Virginia. The report of the Wisconsin State Tax Commission for 1903 discusses similar tables to these given above quite thoroughly and I quote in substance from that report as follows:

The placing of nearly all the states in the first column of the table or in the ad valorem list does not mean that they rely entirely on the old methods of valuation, nor prove a lack of development toward efficiency in the matter of railway taxation. A careful analysis will show the general tendency toward placing the power of valuation in the hands of a central board, although various methods exist.

The general property tax gave good results, and was fairly well suited to the existing conditions when corporations were more or less of a local nature, as they were prior to the Civil War, but the consolidation of the smaller local companies into large systems and trunk lines, which commenced soon after the close of the war, clearly demonstrated the inadequacy of the general property tax as applied to the corporations.

Any departure from the old plan of assessment according to valuation must not be attributed to a changed attitude on the question of the taxation of corporate property on the same basis of valuation as that of individuals; but rather to the transfer of the power to boards having a wider jurisdiction. The trend is toward creating new administrative methods adapted to and capable of determining the valuation of corporate property and transportation companies with greater accuracy.

The result of this development has been to establish methods for railway taxation which are materially different in operation than those employed in the taxation of individuals and most other corporations, and the system or method which has come into most general use is the vesting of the power of valuation in state boards whose powers are sufficiently broad to require reports, examine books, and otherwise obtain information necessary for the fixing of such valuations. The property tax principle is the basis of this method in most states although the mode of procedure often varies, as where the entire property is valued in some instances as against the valuation of only a part, such as the franchises after deducting the real estate, rolling stock, etc., but in the main the provisions are essentially the same.

Roswell C. McCrea, in his report prepared under the direction of the Industrial Commission, clearly outlines the property tax as administered by state boards or officers as follows:

"This (the property tax) is the most common method of taxing transportation companies. In a few cases it is subject to purely local administration, but in most of the states valuations are made by state boards or officials. The workings of the latter method, though not in detail identical in any two of the states, coincide in some such general characteristics as the following: Certain designated officials of the various railroad companies are required to return sworn statements or schedules to state officials, setting forth in detail the length of line with all its tracks, and the proportion thereof in each tax district of the state, all personal property of every kind, all rolling stock, and often a detailed description of the construction of track and roadbed, the time spent in that construction, and the value of materials employed. There is also required a full statement of all real estate owned or used in each tax district; of all stations, houses or other buildings, and all equipment connected therewith; of the amount of capital stock, including its market value, or if there is no market value, the actual value of the shares, in some cases including a list of the shareholders and their places of residence in addition to a statement of the total amount of all indebtedness, generally including current expenses. In some states the schedule must contain a statement of the respective companies' entire gross receipts, entire operating expenses, and entire net earnings, with a supplementary statement of the amount of such receipts, expenses, and earnings resulting from business done exclusively within the state. Neglect to furnish these sworn schedules is generally attended with heavy penalties, and false statements are punishable as perjury. Furthermore, in many cases, the state officials, to whom these reports are made, are empowered to require additional statements when necessary, and even, as provided in a number of states, to summon witnesses, to examine them under oath, and to compel the production of corportion books and papers. The work of assessment on the basis of these returns is generally intrusted to a specially constituted state board, by whom the valuation is determined and in most cases apportioned among the local taxing districts for the computation and collection of the tax. Railroad real estate not directly employed in traffic operations is generally both assessed and taxed by local officials.

"The chief advantages of this general method may be summed up as follows: The duties of assessment are in the main performed by experienced and competent officials, thus minimizing the liability to unequal assessments, as between localities and between companies, under a property tax; the popular demand that corporations be taxed upon the same basis as individuals is realized; the method is in accord with both state and federal constitutional provisions, besides being both reasonably productive and constant in its yield from year to year."

After the assessment is made, as above described, the values are distributed to the local tax districts in proportion to the mileage in each and taxed at the local rate. In assessing roads which cross the boundaries of a state, the assessment board, ascertains the value of the property as a whole and then assigns a value to that part in each state on a mileage basis. This principle of apportionment has been sustained by the courts.

Very great difficulties arise when we try to obtain a measure of value of the franchise. Students of finance and taxation have agreed that there are no less than thirteen separate and distinct methods of taxing corporations which are summed up as follows:

1st. The actual value of the property or the realty added to the personalty, visible or invisible, which is the method employed in the majority of cases.
2nd. The cost of the property, original and improvement.
3rd. Capital stock at par value.
4th. Capital stock at market value.
5th. Capital stock plus bonded debt at market value.
6th. Capital stock plus total debt both funded and floating.
7th. Bonded debt or loans.
8th. Business transacted.
9th. Gross earnings.
10th. Dividends.
11th. Capital stock according to dividends.
12th. Net earnings.
13th. Franchise.

It can be seen that each of the above methods of taxation involve the value of the franchise, except perhaps the cost of the property method, and it is next to impossible to determine in either case what part of the value belongs to the franchise alone. There are many methods adopted in the different states, of determining this value among which it will be well to enumerate a few, such as The value of the Capital stock after the value of the property has been deducted. The value of the stock, less the value of the realty. The value of the stocks and bonds less each or both of the values of realty and tangible property and various others. Each state arriving at its franchise value in its own way without any seeming regard for uniformity, and still more crude and wonderful, often measuring the franchise values of different corporations in entirely different ways.

The real necessity for arriving at a tangible value for the franchise in any case arises from the fact that in a great many of the states the taxes on all kinds of property must be uniform to be constitutional and if it can be shown that the tax is on a franchise instead of on property, the necessity for uniformity vanishes. Again, if the tax is on a franchise it can be upheld as not interfering with interstate commerce and further that while a tax on property is subject to certain exemptions, under the principles of the property tax, a tax on franchises cannot claim these exemptions nor bring forward the claim of double taxation.

It is frequently held by the corporations that the capital stock represents property and that the two are interchangeable and consequently a tax on the property and an additional tax on the stock or the franchise would be double taxation and unjust. A number of the states have decided that the tax on capital stock is not a tax on the property and the federal courts have affirmed that decision, which practically means this, that a tax on capital stock is an excise tax and not a tax on property and therefore does not come under the various state constitutional provisions that the taxes on all property must be uniform.

Some states tax corporations upon the par value of the capital stock; but where it is a common occurrance that the market value of the stock is only a small fractional part, in some instances, and in others many times the par value, it cannot be held that the par value is any index to the real value of the property. Or, if the real value of the stock be taken the situation is unimproved, especially in the case of railways constructed principally from the proceeds of a bonded debt. The profits from a road created in this way may often be enormous and the stock still have a very small value from the fact that the profits are mainly used in paying the interest on the bonded debt and do not become available for dividends on the stock and hence the market value of such stock may be very small. This makes it plainly evident that the value of the stock and the property are in no wise identical and a tax on the capital stock would not be a tax on the whole property.

There seems to be a well grounded interpretation of the principle that no state may levy a tax which will interfere in any way with interstate commerce; but it has been repeatedly held that a tax on the franchise of a corporation is valid, even in cases where the value of the franchise is measured by the gross receipts, part of which may come from the interstate commerce of the corporation, while a tax other than on a franchise tax would be invalid, on account of that interstate business.

It seems to the economist a useless distinction between the value of the franchise and the value of the real property and such distinction is rather legal than economic. Notwithstanding the value of such distinction to the lawyer in trying to evade some constitutional restrictions, to the student of economics the value of the franchise may as well be added to the value of the property for the question of a just tax depends on the ability to pay and is determined by the earning power of the corporation, which is best measured by the business done, minus the expense of doing such business.

Turning to the economics of the different methods of taxation, we see that the general property tax locally assessed was the primitive method in every case and the general tendency is to leave it behind as inadequate and unjust because as levied by local assessors it lacked uniformity.

The method of taxing capital stock at its market value has already been shown to be unjust as in this method the heavily bonded company would entirely escape taxation while in the case of a company without bonds the market value of the stock does not bear any relation to its real value or earning capacity because of the possible manipulations of the stock exchange.

The method of capital stock at its par value has only one point in its favor, that of ease in establishing a base of assessment and is open to all the objections of the previous method with the addition of being more liable to evasion from the fact that only a very small capital may be issued and sold to the stockholders at many times its par value in order to have a small basis for taxation.

THE CAPITAL STOCK PLUS BONDED DEBT.

As to the method of the acceptance of the asset side of the balance sheet for a valuation of the property covered therein. No one in the least familiar with the methods of railway accounting will care to contend that the balance sheet can give anything like a fair valuation of the property as it exists. There is no possible guarantee that the items usually entered as "cost of road" and "cost of equipment" represent anything like the original capital actually put into the road and even if it did, the large fluctuations in the prices of material and labor, during the periods intervening between the times of construction and the present, would render such a statement quite valueless as a determinative of the present value. According to the Reports of the Interstate Commerce Commission the aggregate assets of the railways on June 30th, 1903 was $15,159,097,858 as shown by their balance sheets and no one for a moment would care to say, even after mabing allowances for possible and probable duplication of items, occasioned by intercorporate relation, that the railways of the United States ought to be valued at that figure, consequently we can see that the balance sheet is practically useless for our purposes of valuation.

Capital Stock plus Bonded Debt as a measuring of total values has at different times received the qualified approval of the courts and may be termed the "Stock and Bond Plan" or Rule of the Supreme Court. There are many points to be made in favor of its acceptance and a number against it. The Supreme Court has handed down the following which I quote from State Railroad Tax Cases, 92 W. S. 575 as follows:

"The Statute of Illinois (which is on the Stock and Bond plan) and the rule adopted by the board of equalization, under the power conferred by the clause we have just recited, may not be the wisest mode of doing complete justice in this difficult matter; but we confess we have, on the whole, seen no scheme which is better adapted to effect the purpose, so far as railroad corporations are concerned, of taxing at once all their property, and of making their tax just and equal in its valuation to other taxable property of the State."

Speaking of the market values the Court further says:

"It is therefore obvious that, when you have ascertained the current cash value of the whole funded debt, and the whole cash value of the entire number of shares, you have, by the action of those who above all others can best estimate it, ascertained the true value of the road, all its property, its capital stock and its franchise; for these are all represented by the value of its bonded debt and of the shares of its capital stock."

The Supreme Court further states in the case of Adams Express Co. vs. Ohio, 165 W. S. 185:

,'Now, it is a cardinal rule, which should never be forgotten, that whatever property is worth for the purposes of income and sale it is also worth for purposes of taxation. Suppose a bridge were entirely within the territorial limits of a state, and it appeared that the bridge itself cost only $1,277,000, could be reproduced for that sum, and yet it was so situated, with reference to railroad or other connections, so used by the traveling public, that it was worth to the holders of it, in the matter of income $2,900,000, could be sold in the markets for that sum, was, therefore, in the eyes of practical business men of the value of $2,900,000, can there be any doubt of the state's power to assess it at that sum, and to collect taxes from it upon that basis of value? Substance of right demands that whatever be the real vaule of any property, that value may be accepted by the state for purposes of taxation, and this ought not to be evaded by any mere confusion of words. Suppose an express company is incorporated to transact business within the limits of a state and does business only within such limits, and for the purpose of transacting that business purchases and holds a few thousands of dollars' worth of horses and wagons, and yet it so meets the wants of the people dwelling in that state, so uses the tangible property which it possesses, so transacts business therein that its stock becomes in the markets of the state of the actual cash value of hundreds of thousands of dollars. Does substance of right require that it shall pay taxes only upon the thousands of dollars o ftangible property which it possesses? Accumulated wealth will laugh at the crudity of taxing laws which reach only the one and ignore the other, while they who own tangible property, not organized into a single producing plant, will feel the injustice of a system which misplaces the burden of taxation. * *

"But if the state comprehends all property in its scheme, then the good will of an organized and established industry must be recognized as a thing of value. The capital stock of a corporation and the shares in a joint stock company represent not only the tangible property, but also the intangible, including therein all corporate franchises and all contracts, privileges and good will of the concern. * * * * *

"The value which property bears in the market, the amount for which its stock can be bought and sold, is the real value. Business men do not pay cash for property in moonshine or dreamland. They buy and pay for that which is of value in its power to produce income, or for purposes of sale.

"In conclusion, let us say that this is eminently a practical age; that courts must recognize things as they are and as possessing a value which is accorded to them in the markets of the world, and that no fine-spun theories about situs should interfere to enable these large corporations, whose business is carried on through many states, to escape bearing in each state such burden as a fair distribution of the actual value of their property among the states require."

This rule of the Supreme Court, that is the acceptance of the judgment of the market, in the price it pays for its railroad securities, as a fair measure of value, is a rule of some thirty years standing and many of the reasons which made its application that long ago of value have been reduced or obliterated by the changed conditions under which the great railway properties are now organized and operated. The purchasing of railway securities for investment by individuals is of very small moment as compared with the purchase for purposes of organization and consolidation by Railway corporate powers and syndicates and when it is realized that the conditions which influence the individual in deciding the price he can affard to pay for those securities as an investment, it is readily seen that the market price of railway stocks or bonds is by no means a correct or fair measure of the cash value of those railways, as that phrase is used for purposes of taxation, and is defined by the statutes. As a case in point the recent flight of Northern Pacific stock to the $1000 mark is cited as explanatory of the different conditions which influence purchase. No individual purchaser for purposes of investment could consider such a price and yet for purposes of control it may have been justified.

A second weighty reason why this rule of the Supreme Court fails to satisfy the requirements of a just valuation is the fact that a large majority of the railway stocks and bonds are never bought and sold upon the market and consequently their price is never quoted. In substantiation of this fact some two or three years ago the Interstate Commerce Commission undertook to make a valuation of railway securities on the basis of market quotations and the difficulties which they encountered are best shown by the following quotation from their report:

"It may not be inappropriate, as bearing upon the extent to which reliance can be placed upon the figures submitted, to state the difficulties encountered in this computation * * * The chief difficulty was found in the fact that by far the larger proportion of railway securities are not subject to extensive purchase and sale and on this account fail to disclose the market price. This report deals with over two thousand corporations while the number whose securities were quoted on the stock market in such a manner as to enable a satisfactory computation of the value of the property which they represent in conformity with the rule embodied in the resolution, did not exceed two hundred and twenty-five." Note—the resolution referred to was a resolution passed by the Senate asking for the valuation.

As a conclusion arrived at by the state commission, I quote further from their 17th Annual Report:

"While market valuation of such securities as show a wide market may be of great use in checking values arrived at by other methods or in enabling a correct interpolation of commercial conditions the Commission does not hesitate to say, as a result of this experience that the rule fails to justify a very great degree of confidence in the results to which it leads. If an authoritative and trustworthy valuation of railway properties is to be arrived at, the balance sheet, whether on the side of assets or liabilities, can not be accepted as the starting point for investigation." This method is, however, preferable to any of those previously discussed because it prevents the exemption of the heavily bonded companies.

However, when the tax is on the stock and bonds as well, the revenue will still be inadequate because the tax is applicable only to the bonds owned and held by residents of the commonwealth levying the tax. Further, by the complication of interstate policy the proceeds of the tax on bonds held by non-residents will accrue, not to the state of residence of the bond holder, but to the state wherein the corporate property is held or situated.

TAX ON GROSS RECEIPTS.

The Railroad Tax Commission is usually referred to as the chief authority for defense of the license fee or tax on gross receipts plan and the following is quoted from their report:

"Finally the committee will say that of all the systems of taxation examined by them, those in England, the countries of Europe, and in Michigan and Wisconsin among the states of the Union, seems to them the most intelligent and in conformity with correct principles. The Michigan and Wisconsin principles would seem to be especially commendable. The systems in use in many of the older states on the contrary, and notably in Massachusetts, New York, Pennsylvania and Ohio, are very cumbersome and present hardly any features worthy of study and imitation."

Prof. Henry C. Adams, statistician of the Interstate Commerce Commission is in accord with Dr. E. R. A. Seligman and other economists in their voiced objections to the gross receipts tax and I quote from him as follows:

"The first alternative presented to the financier lies between a gross or a net item as a basis of assessment. If, however, one can confine himself purely to economic considerations, the decision is not difficult. A tax on the gross earnings of a corporation cannot be satisfactory because it takes no account of the operative expenses incident to the securing of such earnings. * * * * It is evident, therefore, that a just assessment as between corporations must take into account earnings as well as expenses."

The license fee presents an advantage to the railroads however, which is denied to the individual property owner in that in good years when the country is prosperous the railroads will have larger gross receipts and even if we still consider the nonexistence of any definite relation between gross receipt and net revenue yet the railroads are better enabled to pay the taxes than they would be in poor years when the gross receipts are less and the expenses possibly as heavy. Consequently they have the benefit of a tax which increases or decreases with their business dependent on the conditions of the money market, crop reports, demand for commodities, etc., which condition or form of sliding scale tax is not enjoyed by any other form of property.

That the fundamental principle of taxation, the application of which all true economists strive to attain, the principle that all taxation shall be just and uniform is openly violated by the gross receipts tax can readily be shown.

In the first place, under this method of taxation all the property outside the right of way of the corporation in question furnishes revenue, the sum total of which gives the gross receipts on which the tax is based, and when this tax is collected and the property outside the right of way is again left open for local assessment and taxation in the civil division in which it is located, we see at once that such property is subjected to double taxation which is unjust as soon as it is shown to be ununiform.

A tax on the gross receipts of any railway in Iowa would be a tax on the receipts derived from all its property in Iowa of every form and a tax on its city property, used as offices or for any purpose outside the right of way, levied and collected for local purposes would be an unjust tax as that property had already paid its just share of taxes through the levy on gross receipts.

The tax on gross earnings was recommended by the railroad tax commission because in their opinion it was least liable to evasion and was the most readily ascertained, but they failed to recognize or seemed to ignore its one fatal defect, that is, that the gross earnings bear no relation whatever to the true earning capacity of the property.

As an instance, a line built on easy curves and minimum grades at an excessive cost must necessarily have large gross earnings to pay a reasonable return on the cost of construction and a tax on the gross earnings would be virtually making the good line pay a tax on the ingenuity of the engineers and the enterprise of the constructors.

Again, two different lines of railway are frequently constructed in the same state but under entirely different conditions, one of them having every natural advantage of location and topography which would necessitate but a minimum amount of the gross earnings to pay a reasonable return on the outlay, while the other may have been forced by necessity to overcome many natural disadvantages of location and construction in reaching the same field of revenue and yet both present the same facilities to the public. It needs no further argument to show that a much larger part of the gross earnings is required to pay a reasonable return on the investment in the second case and consequently a tax on the gross earnings would be imposing a penalty on the more expensive enterprise which is economically, at least, an injustice. As before stated the courts have repeatedly held that an enforced rate which deprives the carrier of a reasonable return on the investment is unjust and a tax which worked the same injustice should also be so held. To avoid these defects several states have introduced a graded gross receipts tax, the rate percent increasing with the earnings. which eliminates in part the objections above stated but not wholly because there is no assurance that the gross receipts bears any particular relation to the net earnings or that a proportional increase in net earnings will naturally follow an increase in gross receipts.

A line of railway so situated as to be under a heavy maintenance and operating expense per mile and whose returns showed gross receipts amounting to $10,000 per mile, might have a smaller net income per mile than a line more favorably situated but having gross receipts amounting to only $6,000 per mile, and the graded gross receipts tax would increase the injustice to the line of high first-cost and maintenance unless it could be shown that the net revenue increased in proportion to the larger gross receipts which is seldom the case.

TAX ON BUSINESS TRANSACTED.

A tax on the business transacted is open to more objections than the gross receipts plan because the amount of business bears no necessary relation to the net revenue or ability to pay.

A business may be extraordinarily large and only nominally lucrative which is often occasioned by fierce competition or poor management and a tax on the amount of business would fail to consider these conditions. The tax on dividends or capital stock according to dividends are economically the same but legally differ materially. An objection raised to this form of tax by economists is that a part of the rightful earned dividends may not be declared but put into a reserve fund and so escape the tax, but as the reserve fund cannot be indefinitely increased but must eventually be turned into the surplus fund which increases the capital, it will finally come under the scope of taxation as increased earnings on the larger capital so the objection is in itself unimportant.

Again, the reserve fund may be turned to improvements or equipment, but as these increase the capital they will eventually return as increased earnings or dividends.

A stronger objection than either of the above arises from the fact that the tax is inadequate when applied to a corporation having a bonded debt because while a company without a bonded debt has sufficient receipts to pay a dividend on the stock of say 10%, another company, with the same earnings, having a bonded debt, would require say 7½% of that dividend to pay the interest on the bonded debt and would have a balance of only 2½% for dividends and the first company would really pay four times as much tax as the latter.

TAX ON NET EARNINGS.

The tax on net earnings is theoretically the most perfect and logical form of taxation and would have the advantage over the gross earnings tax in that it would not discriminate between different corporations or subject the one under the most unfavorable conditions to the highest tax, but it does have objections, which, under our present tax system would be fatal. One is, the possibility of having the net earnings diverted as excessive salaries to the stockholders as officers, or so manipulating the system of accounting that profits would be eaten up and charged as nominal operating expenses, thereby leaving no net earnings to be taxed. A second that under our present system of property taxation it would work an injustice to the holder of other realty in times of depression in that while the holder of real estate or other property not controlled by a corporation is still required to pay a tax for the running expenses of government and the protection afforded him. The corporation would not pay a tax unless their returns would show a net earning which would very likely not be the case as nearly all corporations are required to operate even at a loss in order to avoid a complete dissolution of their property in times of adversity by reason of the fact that a large amount of the corporate property has no practical value except in connection with the immediate purpose for which it was organized. This condition is likewise true of other forms of private property in times of adversity and as the corporation receives a like protection with other forms of realty and is permitted by the government to earn a percentage of what in ordinary conditions is nominally its due, it is plainly just that a tax should be imposed. As the tax on net income would discriminate in favor of corporate property as against personal property or real estate, it is shown to lack uniformity, which is the sacred principle of a just tax.

Under the net earnings tax the corporations pay a bountiful tax in times of prosperity and in times of adversity pay nothing. No other form of property is blessed with such a sliding scale and the owner of real estate or other property outside the corporation must suffer an undue hardship in paying the whole burden of taxation while operating at a loss.

We believe the best method of valuation which receives the approval of the courts is properly termed the "inventory method," and takes into account the fact that the value of railway property exists in two forms, which may be designated as its physical plant and its franchise. Each one of these forms is capable of analysis and subdivision and when thus divided, each element is capable of separate appraisal. The United States Inter-State Commerce Commission has formulated a system of construction accounts and now requires that all improvements and betterments to property be annually reported according to their classification.

That the right to tax franchises, privileges, etc., exists, is held by the courts, and I quote from various decisions as follows:

"State Board of Assessors vs. Central R. Co., 48 N. J. Law, 283, 288, 347. They have a value which can be estimated, and this is the proper duty of the assessor or the board of review. In State Railroad Tax Cases, 92 U. S., 603, it is laid down: 'That the franchise, capital stock, business and profits of all corporations are liable to taxation in the place where they do business, and by the state which creates them, admits of no dispute at this day.' 'Nothing can be more certain in legal decisions' says this court in Society for Savings v. Coite, 6 Wall., 607, 'than that the privileges and franchises of a private corporation * * * may be taxed by a state for the support of the state government.' State Freight Tax Case, 15 Wall., 232: State Tax on Railway Gross Receipts, 15 Wall., 284'. In Wilmington Railroad v. Reid, 13 Wall., 268, the court says that 'nothing is better settled than that the franchise of a private corporation—which in its application to a railroad is the privilege of running it and taking fare and freight—is property, and of the most valuable kind.' And in Morgan v. Louisiana, 93 U. S., 223, it is said that they 'are rights or privileges which are essential to the operations of the corporation, and without which its road and works would be of little value. * * * They are positive rights or privileges, without the possession of which the road of the company could not be successfully worked.'

"The value of franchises, especially in connection with the property reasonably necessary for their exercise and use and which without them would be of little or no practical value, can be estimated as well as the value of other subjects of taxation, though perhaps not as readily or with the same degree of certainty. The cardinal requirement is that, as property, they shall be taxed. All else is matter of method and detail. * * *

"The value of the property of the company consists in the franchises connected with the tangible property, without which the latter would be of little or no value. Hammock v. Loan and Trust Co., 105 U. S., 90; East Ala. R. Co. v. Doe, 114, U. S., 340, 353. The entire property of the corporation, thus considered, is to be regarded and treated as indivisible for the purpose of sale under adverse process; and to permit it to be sold otherwise would defeat the uses impressed on it for public purposes, and which constituted the consideration for the grant of its franchises."

In making a valuation under the inventory method the first step would be to have made by a competent engineer a careful estimate of the value of each of the elements which enter into the entire physical plant of the railway. The physical plant being only a part of the real value of a property, the earning power must also be considered; this earning power is best determined by financiers and the financial rule for valuation is that the worth of a property is determined by what it will produce in actual income. In a case wherein the income is assured by a glance at the past history of the corporation, the risk is less and the real value of the property more in proportion. The earning power is the final and supreme test of value in the opinion of the financiers. Consequently the second step would be to make an analysis of the operating accounts and a study of the history of the property in so far as that would be necessary to determine its condition and business prospects, because an inventory and appraisal of the elements which make a business profitable is just as essential as an appraisal of the elements of its physical plant.

An argument used against this inventory method is the considerable expense attached and that the inventory would in a short time fail to represent the existing condition. In answer thereto it can be stated that the annual reports required by the Interstate Commerce Commission and to the State Railroad Commissioners if made in accordance with the classification of "Operating expenses" and "Construction expenses" and with the established form for the classification and report of earnings, would practically permit of an annual correction of the inventory and thereby an annual declaration of values.

Among the reasons for a careful detailed valuation of the railway properties of the United States there are two which stand out as worthy of particular mention. In the first place, the rulings of the courts for the determination of reasonable rates for freight and passenger traffic, as handed down to us through their decisions, lay particular stress upon a fair value of the roads where rates are under complaint. It is not essential or a part of this paper to cite a list of the particular decisions setting out this position but it is sufficient to state that no tribunal to whom the question might be submitted could pass a satisfactory judgment upon the reasonableness of particular rates without taking into consideration the just valuation of the property whose rates are questioned. This is obvious from the facts that the courts have repeatedly held that an enforced rate which is so low as to deprive the carrier of its property, or in other words, a denial of the right to a reasonable return upon the capital invested is contrary to the Constitution. Of course many other matters must also be considered in deciding upon the justice of any rate such as the proper adjustment of schedules, similarity of conditions, etc., but including all of these is the idea that the rate which is reasonable and fair to all concerned is the one which gives a reasonable return upon the capital invested in property engaged in the service of transportation.

The second reason is scarcely less important and arises from the fact that the subject of reasonable railway taxation is very closely related to the subject of reasonable railway rates, and the general fact that the value of corporate property is made the basis of public contributions is shown from the table, "analysis of taxes, by states," given in another part of this paper.

So long as a state exercises the right of restricting the rates of the railways they must also exercise a protective right as well, because the right of sovereignty is not simply coercive but is also protective. When a government by its restrictions reduces the rates and thereby the income from the property, the courts have held that they must likewise reduce the taxes imposed if the income does not still produce a reasonable return upon the capital invested in transportation. This brings us to the question of what is the actual amount of capital invested in any transportation enterprise and we will need to follow through the thread of actual construction cost to learn whether the sum total of the stock and bonded debt is a fair measure of the capital really invested in the enterprise. Railroads are usually constructed in the following manner:

A few persons, five or ten in number who are usually not capitalists, associate themselves for the purpose of forming a corporation and securing a franchise or a charter to construct a line of railway between two previously selected points of vantage.

Money is subscribed or borrowed sufficient to pay for preliminary surveys over two or more slightly different lines between the selected terminal points of the proposed road.

The people adjacent to each of the proposed routes become interested in having the line pass near their towns, lands or places of business and frequently give a right of way through their lands in addition to the liberal donations of money in order to secure the location of the road on one of the specified lines. The road is usually located on the line, other things being equal, which makes the most liberal donations of land and money. The promoters then have little difficulty in raising enough money by collecting the donations or borrowing money on them to build the first section of the line, usually only a few miles in length, which they at once bond for more than the cost of the completed section. The capitalist then appears and after satisfying himself that the interest on the bonds will be paid when due, purchases them at a liberal discount on their face value and the proceeds from this bond sale are converted into a second section of the line which is again bonded for sufficient sums to continue the construction of the road.

When the road is finally completed it will be found to have been bonded for more than the cost of construction and everything has been paid for without drawing more than a nominal sum from the pockets of the original stock holders.

If, as required by the laws of some states, a percent of the original capital stock has been paid in, the money is retained in the treasury and used in paying the first annual dividend.

The face value of the stock per mile of road is usually about equal to the face value of the bonds which have already been issued and very few cases will be found upon examination, to show that the original stock holder had sufficient means to build and equip the line without the issuing of bonds to nearly the full actual cost of construction. These bonds then represent practically the whole original cost of construction in all save a very few cases and the stock issued does not represent anything but a very small portion of this original cost.

This stock is issued by the promoters to themselves and does not represent a value which in any manner benefits the public and it is unjust that the public should be required to pay rates of transportation which will insure dividends upon such stock. The financiers, whom the courts say are the ones most eminently fitted to judge of such matters, are satisfied with the interest paid on the bonds else they would have refused to buy them and in no case can it be shown to be just to demand dividends from the public upon stock when the bonds issued covered the cost of the road.

As to the best method of finding the actual value of the road upon which it is fair and just to demand dividends from the public in the rates which they are required to pay, attention is called to the methods employed by the State of Texas persuant to the requirements of a law enacted by the Texas State Legislaturwe in 1893, which is given in part as follows:

"The Railroad Stock and Bond Law, as published under Title XCIV, Chapter 14, Revised Statutes of Texas, 1895, declares:

'Article 4584 a.,—Among other things, the power and authority of issuing or executing bonds, or other evidences of debt, and all kinds of stock and shares thereof, and the execution of all liens and mortgages, by railroad corporations in this State, are special privileges and franchises, the right of supervision, regulation, restriction, and control of which has always been, is now, and shall continue to be vested in the State Government, to be exercised according to the provisions of this and other laws.

'Article 4584 b—Hereafter no bonds or other indebtedness shall be increased or issued or executed by any authority whatsoever, and secured by lien or mortgage on any railroad or part of railroad or the franchises or property appurtenant or belonging thereto, over or above the reasonable value of said railroad property; provided that in case of emergency on conclusive proof shown by the company to the Railroad Commission, that public interests or the preservation of the property demand it, the said Commission may permit said bonds, together with the stock in the aggregate, to be executed to an amount not more than fifty percent over the value of the said property.'

It further provides, with regard to the railroads constructed and in operation at the time of the passage of the law:

'Article 4584 c—It shall be the duty of the Railroad Commission to ascertain and in writing report to the Secretary of State, the value of each railroad in this State including all its franchises, appurtenances and property. After it shall have prepared said report of value, the Commission shall give the company interested ten days' notice in writing, by registered letter to the president, treasurer or receiver of said railroad, to the effect that said report is ready to be made, and that if it have any objections thereto it must file them in writing, within forty days after said service, or the same will be so deposited with the Secretary of State as correct. Should the company or its duly authorized representative file with said commission any objections to said report of value, the Commission shall duly investigate and pass on the same. On investigation, if the Commission conclude that its report of value is too low or too high, then it shall make the necessary correction before filing it. Should no objections be filed within the time permitted, or being filed and on examination found without merit, the Commission shall forthwith file its said report in the office of the Secretary of State, where it shall remain as a public record, as a limitation for the issuance of indebtedness under the limitations prescribed in Article 4584 b. To promote public interests and protect private rights, the Commission, after due notice under the rule herein prescribed, may correct its report of value of any railroad at any time it may deem proper.'

With respect to railroads constructed after the passage of the law, or existing at that time, but against which stocks and bonds were not outstanding in excess of their value, the following section defines the method of procedure to be had in making the application for issuance of stocks and bonds, and defines the basis on which the Railroad Commission must make its valuations.

'Article 4584 f—Should any company or corporation authorized to construct, own or operate a railroad in this State, desire to issue bonds or other indebtedness, to be secured by lien or other mortgage on its franchise and property, in advance of the completion of the said railroad, it shall make application to and first procure the consent of the Railroad Commission thereto. In said application it shall exhibit to the Commission its contract with the construction company, if it have any; the profile of its completed road or part of road, the evidence of its right of way, depot grounds, terminal facilities; the extent and value of work done or in process of completion; the amount of property received; the amount of stock subscribed and the amount paid in; and all other necessary facts showing the value of the franchises and property proposed as security for said contemplated debts. If, on investigation, the Commission is satisfied that the company is acting in good faith, and that its contract with the construction company is reasonable and fair to the public, then it shall authorize the execution of said indebtedness and lien to the extent necessary for the demands of the work, at no time to be more than fifty per cent. over the value of the whole property and franchises.'

After authority has been obtained from the Commission to issue stocks and bonds upon a railroad, the same or any portion of it which has been completed, Articles 4584 h and 4584 i of the Statute, which is not necessary to quote here, prescribe the method of procedure to have the stocks and bonds registered in the office of the Secretary of State. It is only after such registration that the securities become valid obligations against the property which is mortgaged. Stocks and bonds are approved and authorized to be registered by the Commission only upon and to the actual value of the completed railroad, as the same is determined by inspection."

Under this law the Railroad Commission of Texas has appointed engineers to assist in making these valuations who have gone personally over the ground on foot, profiles in hand and made a detailed inspection of the roads as they exist and affixed values thereto. The quantities of excavation and embankment where original records could not be obtained were approximated from the profiles or from measurements made upon the finished road bed, and the classification was determined by inspection. Bridges were estimated by actual measurement where original cost records were not obtained. Extent of Right of Way, depot and terminal grounds were obtained from city and county tax records or furnished by the railroad companies.

After a complete and careful examination of the line by the engineers, detailed estimates were prepared on sheets of the following form:

STIMATE SHEET: "A," ROADBED, STRUCTURES, ETC. 1134-304-2½m

RAILROAD COMMISSION OF TEXAS

__________Rail______, Mile No.______From__________

ITEM	Estimate Upon which Authority to Issue Stock and Bonds is Issued Made........................190..					Estimate for Registration of Stock and Bonds Made..............190..				Estimate for Registration of Stock and Bonds Made..............199..			
	Rate	Unit	Amo'nt	Value		Rate	Amo'nt	Value		Rate	Am'nt	Value	
ight of Way		Acre											
epot Grounds		Acre											
learing & Grubbing		Acre											
arth Excav		Cu. yd.											
ravel Excav		Cu. yd.											
oose Rock Excav		Cu. yd.											
olid Rock Excav		Cu. yd.											
arth Embank		Cu. yd.											
auled Embank		Cu. yd.											
ip Rap		Cu. yd.											
oundation Excav		Cu. yd.											
oundation Piling		Lin. ft.											
rillage		B. M.											
asonry Piers		Cu. yd.											
ylinder Piers		Lin. ft.											
asonry Culverts		Cu. yd.											
teel Trusses		Lb.											
late Girdles		Lb.											
restle Timber		B. M.											
restle Piling		Lin. ft.											
rains, Box													
rains, Pipe		Lin. ft.											
attle Guards		Each											
encing		Mile											
oad Crossings													
igns													
ies													
ie Plates		Each											
teel Rails		Each											
oints Complete		Ton											
pikes		Each											
aying & Surfacing		Keg											
idings		Mile											
witch Furniture		Lin. ft.											
ailroad Crossings		Se*											
nterlockers													
allast													
elegraph													
assenger Depot		Mile											
reight Depot		Each											
otton Platform		Each											
ection House		Sq. ft											
ater Station		Each											
tock Pen		Each											
oal Chute		Each											
rack Scales		Each											
rack Scales		Each											
Total													

Mile No.______From__________ __________Rail______

ESTIMATE SHEET: "B," ROLLING STOCK.
ESTIMATE SHEET: "C," MISCELLANEOUS ITEMS.

RAILROAD COMMISSION OF TEXAS

_______________ Rail _______

ITEM	Estimate Upon Which Authority to Issue Stock and Bonds is Issued				Estimate for Registration of Stock and Bonds				Estimate for Registration of Stock and Bonds			
	Made..............190..				Made..............190..				Made..............199..			
	No.	Rate	Value		No.	Rate	Value		No.	Rate	Value	
Passengers Locomotives												
Freight Locomotives												
Switch Locomotives												
Passenger Cars												
Chair Cars												
Baggage Cars												
Express Cars												
Postal Cars												
Combination Cars												
Officers' Cars												
Box Cars												
Flat Cars												
Stock Cars												
Coal Cars												
Tank Cars												
Refrigerator Cars												
Water Cars												
Hands Cars												
Push Cars												
Total "B"												
"C" MISCELLANEOUS:												
Road Tools												
B. & B. Tools												
Machine Tools												
Machine Shops												
Machinery												
Round Houses												
Turntables												
Furniture, Etc												
Stationery												
Materials and Supplies												
Terminal Grounds												
Franchises												
Charter Fees, Etc												
Legal and Engr. Expenses												
Interest during Constr												
Total "C"												
Totals "B" and "C"												

_______________ Rail

The values applied to the several items making up the estimate of construction cost were fixed equal to the current market (contract) price plus a liberal allowance for contingencies and the values applied to real estate and Right of Way were fixed according to the current market price of adjoining property, making no deduction for land donated or acquired for less than its market value. Grading quantities were estimated at the outside (contract) price and materials at the market quotations plus the freight rate to the point of construction and a liberal distribution charge. No deduction was made for depreciation of any article making up the road bed and structures and no additional amount was added for the value of a seasoned road bed.

Franchises were not valued except municipal franchises and then only where the streets of a city or town were occupied by the tracks of the railroad in lieu of the purchase of adjoining property when the valuation was placed at about 33 per cent of the value of improvements thereon. 5 per cent to 6 per cent of the total estimated value of the railroad was allowed to cover the items of legal and engineering expenses and a like amount for interest during construction.

Since the passage of the Stock and Bond law the Railroad Commission has valued some 10,349 miles of railroad in the state of Texas and the list as completed up to October 20, 1903, is shown in the following table:

TABLE NO 1

NAME OF RAILROAD	Mileage	Valuation	Value Per Mile
Beaumont Wharf and Terminal	3.08	$ 179,481	$ 58,273
Cane Belt	68.69	830,968	12 097
Chicago, Rock Island and Mexico	91.75	1,475,361	16,080
Chicago, Rock Island and Texas	147.29	2,760,423	18,471
Dallas, Cleburne and Southwestern	9.77	155,917	15.959
Denison, Bonham and New Orleans	24.11	375,266	15,565
Eastern Texas	30.41	454,528	14,947
Fort Worth and Denver City	454.13	5,771,582	12,709
Fort Worth and Rio Grande	143.10	2,003,839	14,003
Galveston, Harrisburg and SanAntonio	919.06	16,142,298	17,564
Galveston, Houston and Henderson	48.85	1,527,023	31,259
Galveston, Houston and Northern	56.36	1,000,000	17,743
Gulf, Beaumont and Kansas City	67.55	676,485	10,015
Gulf, Beaumont and Great Northern	58.22	853,123	14,653
Gulf, Colorado and Santa Fe	987.34	16,649,746	16,863
Gulf, Western Texas and Pacific	109.67	1,318,082	12,019
Hearne and Brazos Valley	16.42	106,624	6,493
Houston and Texas Central	690.42	13,683,834	19 820
Houston, East and West Texas	190.69	2,042,693	10,712
International and Great Northern	1,049.93	19,928,547	18,981
Missouri, Kansas and Texas of Texas	1,069.83	16,688,500	15,599
New York, Texas and Mexican	122.93	1,464,206	11,911
Orange and Northwestern	32.80	458,793	13,987
Paris and Great Northern	16.18	288,718	17,844
Pecos River	54.13	392,405	7,249
Red River, Texas and Southern	57.49	1,215,332	21,140
Rio Grande	22.17	310,551	14,008
Rio Grande and Eagle Pass	26.89	234,695	8,729
Rio Grande and El Paso	20.15	481,824	23,912
St. Louis, San Franciso and Texas	6.74	407,454	60,453
St. Louis, Southwestern of Texas	652.41	10,057,889	15,706
San Antonio and Arkansas Pass	687.67	8,677,698	12,619
Southern Kansas of Texas	114.95	1,190,672	10,358
Sugarland	14.12	109,415	7,749
Texarkana and Fort Smith	79.24	1,603,789	20,240
Texas and Louisiana	21.91	203,688	9,297
Texas and New Orleans	433.14	8,667,796	18,896
Texas and Pacific	1,039.33	17,730,689	17 060
Texas Central	215.36	2,885,256	13,397
Texas Mexican	162.40	1,457,638	8,976
Texas Midland	112.49	1,914,578	17,020
Texas, Sabine Valley and Northwestern	38.41	398,848	10,384
Texas Southern	68.69	743,980	10,831
Velasco, Brazos and Northern	20.06	240,256	11,997
Weatherford, Mineral Wells and Northwestern	23.00	327,496	14,239
Wichita Falls	17.96	242,838	13,521
Wichita Valley	51.36	486,768	9,478
	10,348.75	$166,819,992	$ 16,120,00

The reduction in the average indebtedness of the railways of Texas since the law went into effect is well shown by table No. 2.

TABLE NO. 2

ON JUNE 30	Miles of Railway in Operation	Stock Outstanding—Per Mile	Bonds Outstanding—Per Mile	Total Stocks and Bonds Outstanding—Per Mile
1894	9,154	15,076	25,726	40,802
1895	9,291	14,874	25,420	40,294
1896	9,437	14,647	25,302	39,949
1897	9,484	14,320	24,793	39,113
1898	9,540	14,205	24,036	38,241
1899	9,702	13,997	23,562	37,559
1900	9,867	13,724	23,202	36,926
1901	10,154	12,922	22,649	35,571
1902	10,617	11,971	21,779	34,167
1903	11,029	11,971	21,464	33,435
Total reduction for 9 years		3,105	4,262	7,367
Average reduction per year for 9 years		345.00	473.55	818.55

The fact that the valuation and taxation of the franchise was omitted by the Stock and Bond law of Texas is an excellent recommendation to the purchaser of transportation. The franchise is no real part of the road, its existance is called forth and firmly established before the project is well under way or a shovelful of earth turned in actual construction. It is simply a gift of the public for the public good and there is no just reason why it should have a capitalized value for the purpose of demanding revenues thereon from the public who buy the transportation.

It is the fault of the public which places this weapon in the hands of the corporations for forcing them to pay dividends on fictitious values. In their wild scramble to tax everything in possession of the corporation they obtain a few thousands of dollars in taxes and are forced by the courts to pay dividends on this increased valuation out of the price they pay for their transportation because of the principle that all corporations are entitled to a just return upon the valuation of property employed in their business. And when a franchise is valued by state authorities for purposes of taxation, this value immediately attaches itself to and becomes a part of the value of the corporate property, by giving it, in addition to its right and privilege of earning a just return upon its own value, a right to earn a reasonable return upon the value of the franchise as fixed by the state authorities for taxation purposes, which may result in an increase in rates or prevent a demand for a decrease in rates which could be shown to be unreasonable upon a proper valuation, for when the producer demands a lower rate the corporation answers that the rate is already so low that it just earns operating expenses and a reasonable return upon the value of the property.

The producer in turn states that the road of the corporation in question is worth but a certain sum per mile, stating the value of the physical property. "That may be true," states the corporation, "but you have neglected to take into consideration the value of our franchise which for taxation purposes you have valued at $2,000,000, taking into consideration the fact that you value private property at 1-3 its actual value for taxation purposes, this would fix the value of our franchise at $6,000,000 and the courts of the land allow us a gross earning, of 12 per cent as a reasonable return upon the valuation."

The Railroad Commission is appealed to and attempts to lower the rates when the corporation organized under the laws of another state gets out a restraining order through the United States courts and when the case comes to trial the Railroad people testify that the road is worth the real physical value plus the value of the franchise and the state authorities, having no expert testimony to the contrary, the decree is granted and the restraining order made perpetual. The State then pays the cost of the suit, and not only that, but the cost of opposing counsel, as well for the corporation attorneys' fees are turned in as operating expenses and the public pays them in the next transportation it buys.

In taxing the franchise for this valuation of $2,000,000 the state probably gets 2 per cent on that money or $40,000 paid in by the corporation in the form of taxes, and for the purpose of fixing rates the corporation values its franchise at $6,000,000 and makes its rate sufficient to produce 12 per cent gross receipts on that amount or $720,000 a balance of $680,000 in favor of the corporation and the public has virtually paid $720,000 in increased rates for the purpose of getting back $40,000 in taxes.

In conclusion the author is willing to admit the more than ordinary difficulties to be encountered in a subject of such scope and over which legislators and executives have blindly stumbled in their search for equal justice to all. He will be greatly pleased to have pointed out to him the points in which he has erred and doubtless there are many; but in substantiation of his acceptance of the "inventory method" of valuation for a basis of railway taxation as being the fairest and most acceptable system yet devised, the following quotations are given from the leading engineering periodicals of the world as evidence of the way in which the Texas Stock and Bond Law is viewed by these authorities. The first quotation is from the Engineering News and American Railway Journal for April 23, 1896.

"As a matter of fact, after a careful examination of the last report of the Texas Railway Commission, we are constrained to believe that this bad name is undeserved. If the attitude of the State towards the railways is correctly set forth in the Commission's report, we are free to say that the position is economically and politically correct, and that other States and State Railway Commissions can profitably study the work that has been done in Texas.

"It may be said at the outset, that the Texas Commission has attempted a task which has never been attempted before anywhere, and that is to base railway rates upon the cost of service. The position of the Commission is that the railways are entitled to charge rates high enough to return a fair rate of interest on the actual value of their property, and no higher. By value is meant not the par value of outstanding stocks and bonds, nor the amount of money which has been actually spent and charged to capital account; but it is what the entire plant of the railway company could be duplicated for at the present time.

"That this principle is a sound and safe one, and equitable to both the railways and the public, all careful students of the railway problem will generally agree. That the Texas Commission is willing to adhere to it, and carry it out impartially, the following extract from its report bears witness:

" 'While it is the duty of this Commission, as far as it has the authority to do, by its rates and rulings, to protect the public against excessive freight charges, it is equally its duty to establish and maintain such rates as will secure to the railway corporations a proper return of interest on the capital invested, if the conditions of the roads with reference to population and busniess will enable it to do so. And it would not hesitate, if it should be shown to be necessary, to increase rates for this purpose. This Commission has often stated to the freight agents and Traffic managers, in its meetings with them, that if the railway companies engaged in interstate shipment would make and maintain rates which would be fairly compensatory to them on such shipments, this Commission would do all in its power, by its rates, to secure them reasonable revenue on their railway investments in this State. And we now repeat that statement. But this suggestion contemplates good faith on both sides in the making and maintenance of rates.'

"That the principles above stated have been carrier out in practice, as far as the Commission is concerned, appears to be shown by the figures in the report; and it is stated that, notwithstanding the important reductions in freight rates (some of which were made by request of the railway companies), the returns show an increase in revenue. Rates are not always lowered by the Commission, but in some cases are as high as—or higher than— those which were in force before the railways were subject to regulation. This has been on account of the necessity of preserving rates which have long been in force on account of competition or other local conditions. These cases are quite limited in number. There has been steady increase in both gross and net earnings per mile in Texas, as compared with the equally steady increase for the whole United States.

"Under the Texas law, no bonds can be issued on any railway line for a greater amount than the actual value as determined by the Commission. Anyone familiar with the water that has been injected into the capitalization of almost every railway corporation in the country at some time in its history, through bond issues, will confess that Texas is attempting to remedy a very real and serious evil; every one will watch with interest to see what measures of success is attained in the work that she has undertaken."

The following is from "Engineering" the London Contemporary of the Engineering News:

"In Texas a Railroad Commission, with unusual powers, has been organized, through the legislative exertion of ex-Governor J. S. Hogg, and the annual report before us is interesting. Their ruling as to rates has been affirmed by the Supreme Court; but the chief point in this connection is their function in reference to the promotion of lines and the issue of bonds. Railway companies have a happy way of meeting liabilities by the issue of bonds in excess of the worth of their property, and one duty of the Commission is to value the property and franchises, and thus determine the maximum amount of bonds that may be issued. The estimate is based on the cost of construction plus a reasonable profit to contractors. Thus 6,000 miles or so have already been valued, and the average worth is £3,185 per mile, beyond which the property cannot be bonded. With this restriction there is a greater chance of profit to the share or bond-holders. Again, the Commission has power over new railways. The route proposed is considered in the light of probable traffic, and is laid out also to involve the least difficulties. It is suggested that otherwise the interested promoter and constructor would map out the most difficult route, so that his profit on construction might be the greater. Bonds for new lines in Texas can only be authorized on the basis of the cost of constructing the road of a certain grade, with a certain kind of material, and of a certain standard of excellence to insure the minimum expense of working. The new legislation has been opposed on the plea that the effect is to discourage the construction of railways and the investment of capital in this class of security. But, as a matter of fact, the legislation, which has been in force four years, tends to encourage investment, as it insures the carrying forward of lines which have a prospect of traffic and at a fair cost of construction, discouraging speculative lines such as we have incidentally referred to. Moreover, it insures very reasonable rates of freight. It is the guarantee which is given by this legislation that is necessary to reassure the investing public in all the State railways."

Engineering Experiment Station

IOWA STATE COLLEGE

Ames, Iowa

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BULLETIN
VOL. III No. 3

Mechanical Engineering Section

Notes on Power Generation in Iowa

DECEMBER, 1905

...Introduction...

In accordance with the expectation expressed in the introduction to "Notes on Steam Generation with Iowa Coal" published in September, 1904, this bulletin presents the results compiled from the college and other records, of recent investigations of interest to power users in Iowa.

G. W. Bissell.
W. H. Meeker.

Iowa State College,
December, 1905.

Notes on Power Generation in Iowa

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Part I

*U. S. GEOLOGICAL SURVEY TESTS OF IOWA COALS

These tests included

1. Chemical Composition.
2. Calorific Value.
3. Washing Tests.
4. Coking Tests.
5. Briquetting Tests.
6. Evaporative Boiler Tests.

Five Iowa coals were used, viz:

TABLE I.

Mark	Operator	Mine	Description
Iowa 1	Anchor Coal Co. Ottumwa	No. 2 Laddsdale	Middle Bed. Over 1¼ inch Screen.
Iowa 2	Mammoth Vein Co. Hamilton	No. 5 Liberty Twp. Marion Co.	Big Vein. Run of Mine.
Iowa 3	Gibson Coal Mining Co., Des Moines	No. 4 Altoona Polk Co.	Third Seam Over 1⅜ inch Screen.
Iowa 4	Centerville Block Coal Co., Centerville	No. 3 Centerville Appanoose Co.	Lower Bed. Over 1⅜ inch Screen.
Iowa 5	Inland Fuel Co. Chariton	No. 1 Chariton Lucas Co.	Lower Bed. Run of Mine.

A car load of coal from each mine was delivered to the testing plant and from these the coal used for the several tests was taken. In addition "mine samples" were taken for the chemical and calorific tests. All samples were collected and the car loads shipped under the supervision of an agent of the Geological Survey.

1 and 2. Chemical Composition and Calorific Value. The results of tests of samples from car loads are shown in Table II.

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*Abstracted from Bulletin No. 261, U. S. Geological Survey. "Preliminary Report on the Operations of the Coal Testing Plant of the United States Geological Survey at the Louisiana Purchase Exposition, St. Louis, Nov. 1904." The Iowa coals used in these tests were selected by the Iowa Geological Survey, F. H. Wilder, Director, by whom also all necessary arrangaments were made for delivering the car load lots at St. Louis, free of expense to the U. S. Geological Survey.

TABLE II.

Samples	Iowa 1	Iowa 2	Iowa 3	Iowa 4	Iowa 5
*Moisture Percent.......	8.24	14.21	13.88	14.08	15.39
Carbon, Volatile "	30.74	33.17	36.94	35.59	30.49
Fixed "	45.02	37.40	35.17	39.37	41.49
Ash "	16.00	15.22	14.01	10.96	12.63
	100.00	100.00	100.00	100.00	100.00
Sulphur "	5.03	4.66	6.15	4.26	3.19
**Calorific Value British Thermal Units...............................	11027	10019	10244	10723	10242

3. *Washing Tests.* The ash and sulphur contents of the Iowa coals before and after washing are set forth in Table III.

***TABLE III.

Sample	Iowa 1	Iowa 2	Iowa 3	Iowa 4	Iowa 5
Ash in Car Sample, Percent......	16.00	15.22	14.01	10.96	12.63
" " Washed Coal "	10.25	10.28	8.03	7.14	7.93
" removed by Washing"	5.75	4.94	5.98	3.82	4.70
Sulphur, Car Sample "	5.03	4.66	6.15	4.26	3.19
" Washed " "	4.61	3.93	4.55	3.59	2.28
" removed by Washing	0.42	0.73	1.60	0.67	0.91
Weight of Coal Washed, Tons	5	5-½	4-½	4-½	5

In this connection the washing tests on a slack coal from Putnam County, Missouri, are of interest and the results thereof with comments are herewith presented *verbatim* from Bulletin No. 261.

Missouri No. 3. "This sample consisted of a car load of "slack from the Mendota Coal and Mining Company, operating at "Mendota, Putnam County, Mo. Like most of the coals from "northern Missouri and southern Iowa, this coal contains a heavy "percentage of impurities, and the operators are very desirous of "improving their product It was for this reason that the above "test was undertaken, as the results of this test will doubtless ap-

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*The percentages of moisture, etc., are for the coal in the condition of the samples as taken from the car upon arrival in St. Louis. See Bulletin No. 261, U. S. Geol. Survey, p. 30 and 41-43.

**By calorimeter. Idem. p. 31.

***See Bulletin No. 261, pp. 68-70 incl.

"ply to the entire field in Putnam County, Mo., and Appanoose "County, Iowa.

"This sample, consisting of 14,000 pounds of slack, was "passed through rolls having an aperature of 1 ¼ inches and then "washed through the modified Stewart* jig. After the coal was "crushed it was sampled and analyzed, giving the figures shown in "the column marked "Raw Coal" in Table IV. After washing it "was again sampled and analyzed, with the results given in the "second column of the table. The refuse was also sampled and "analyzed, with the results shown in the third column.

TABLE IV.

ANALYSIS OF MISSOURI NO. 3 COAL AND REFUSE, SHOWING EFFECT OF WASHING.

	Raw Coal	Washed Coal	Refuse
Moisture	14.37	23.90	23.38
Ash	28.39	7.59	36.70
Sulphur	4.30	2.89	3.94
Weight, in pounds	14,000		4,931
Calculated weight of coal, in pounds	8,013	6,045	1,968

"The refuse weighed 4,931 pounds. It appears from the "chemical analysis that the raw coal contained 8,013 pounds of "coal exclusive of moisture and ash, and the refuse 1,968 pounds. "The washed coal, therefore, must have been the difference, or "6,045 pounds.

"The results given above show great improvement. The "slack was extremely dirty, as the analysis shows. Washing re-"duced the ash from 28.39 per cent in the raw coal to 7.59 per "cent in the washed coal. The sulphur was likewise reduced from "4.30 to 2.89 per cent. On the whole the reduction in the im-"purities is highly satisfactory, but the amount of coal lost in the "operation (1,968 pounds) is too great for economical work. It is "possible that better results might have been obtained if the tests "had been continued, but enough has been done to show that "great improvement may be made in the quality of the slack coal "in this field by washing.

"The improvement in washing this slack was still further

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*Manufactured by the Link Belt Machinery Co., Chicago.

"tested by steam tests made on the raw coal and also on the "washed product.

TABLE V.

STEAM TESTS SHOWING EFFECT OF WASHING MISSOURI NO. 3 COAL.

Coal Tested....	Total coal consumed	Horse-power developed by boiler	Dry coal burned per sq.. ft. of grate surface per hour	Equivalent evaporation from and at 212° F, per pound of dry coal	Dry coal, per indicated horse-power	Dry coal per electric horse power
	Pounds		Pounds	Pounds	Pounds	Pounds
Raw slack....	10,828	149.5	21.85	5.82	4.86	6.00
Washed slack....	11,044	189.7	21.72	7.43	3.81	4.70

"A washing test was also made of about 6½ tons of this coal "for cooking purposes. No trial was made of the raw coal in the "oven, so it is impossible to make a direct comparison of the coke, "but the chemical analyses give some idea of the improvement "produced."

TABLE VI.

ANALYSES SHOWING EFFECT OF WASHING MISSOURI NO. 3 COAL.

	Raw Coal	Washed Coal for Coking
Ash	28.39	7.24
Sulphur	4.30	2.74

**4. Coking Tests.* The washed Iowa coals were subjected to coking tests under conditions and with effects exhibited in Table VII.

TABLE VII.

	Iowa 1	Iowa 2	Iowa 3	Iowa 4	Iowa 5
Weight of washed coal, lbs	9,500	10,000	8,000	8,000	9,000
Time of Burning, hrs	46	64	43	40	66
Weight of Coke, lbs	4,828	3,866	3,336	3,722	(not completely burned)
Weight of Refuse, lbs	572	1,153	585	585	

All resulting coke was too high in sulphur for blast furnace use, and the large ratio of ash to fixed carbon in the coal makes the coke-product relatively small.

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*Bulletin No. 261, pp. 124-125.

**5. *Briquettlng Tests.* Iowa No. 4.

"One ton of this coal was briquetted with 7 per cent of pitch "E. The briquettes were well pressed, of a grayish color, but on "cooling crumbled decidedly. They weighed 6.73 pounds each. "As they did not contain an excess of pitch, 7 tons more of this "coal were briquetted with 8 per cent of pitch E, in order to have "a sufficient quantity for a steam test. The resultant briquettes "were bluish black in color, but they were not quite hard enough, "although fairly strong, and would stand considerable hard treat-"ment in transportation. In burning they held together until "consumed. They weighed, on an average, 6.77 pounds each. "The eggettes made from this same mixture were stronger than "the briquettes, had a polished surface, but were very brown in "color. In the cook stove they burned very satisfactorily without "crumbling hardly at all."*

6. *Evaporative Boiler Tests.* The boiler tests of the U. S. Geological Survey were made on Heine water tube boilers of which two were available, being duplicates and having the following governing proportions,

****TABLE VIII.**

Item No.	Item	Dim'nsions
1	Rated capacity of boiler .... horse-power	210
2	Water-heating surface .... square feet	2,031
3	Superheating surface	None
4	Grate area .... square feet	40.6
5	Kind of draft	Natural
6	Height of steel stack .... feet	115
7	Area of steel stack .... square feet	7.67
8	Number of tubes	116
9	Outside diameter of tubes .... inches	3.5
10	Steam pressure .... pounds	85
11	Observations taken every 20 minutes	
12	Approximate duration of each trial .... hours	10

*"The steam generated in the boilers was used chiefly by an "Allis-Corliss engine of 250-horse power capacity, and the power "of the engine was absorbed by a 200-kilowatt direct current, 240-"volt Bullock generator, to which the engine was belted.

"The engine was the simple non-condensing type. The "cylinder was 22 inches in diameter and the stroke was 42 inches. "The engine ran about 60 revolutions per minute.

"Numerous tests made to determine the steam consumption "of the engine, as well as the mechanical efficiency of the engine "and generator together, gave the following average results:

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*Verbatim. Bulletin No. 261, pp. 158-159

**Bulletin No. 261, p. 75.

AVERAGE RESULTS OF ENGINE AND GENERATOR TESTS.

Number of pounds of steam used per hour per indicated horsepower developed by the engine 23.6

Mechanical efficiency of the engine and generator combined (per cent)........ 81

"From these figures has been calculated the electrical horse-"power delivered to the switchboard for such tests of the boilers "as were not accompanied by a test of the engine and generator."

The general results of the boiler tests are given in Table IX which includes also corresponding results of a test with briquettes made from Iowa Coal No. 4.

TABLE IX.*

SUMMARY OF U. S. GEOLOGICAL SURVEY EVAPORATIVE TESTS OF IOWA COALS.

	Iowa 1	Iowa 2	Iowa 3	Iowa 4		Iowa 5
				Coal	Briquette	
Analysis of Fuel**						
Carbon fixed pr ct.	39.89	33.73	35.77	37.28	37.85	38.83
" volatile "	33.08	35.35	36.14	34.09	36.50	31.76
Moisture "	8.69	14.88	12.44	13.48	13.24	16.01
Ash "	18.34	16.04	15.65	15.15	12.41	13.40
Sulphur, separately determined	6.39	4.73	6.07	5.04	3.90	3.09
Duration of trial, hrs	10.02	9.92	10.03	10.00	10.03	9.98
T't'l coal cons'm'd lbs	10,331	10,986	10,668	9,385	9,900	11,200
Horse power developed by boiler	197.7	192.7	189.3	167.3	184.5	204.7
Rate of combustion, dry coal per square foot grate surface per hour, lbs...........	23.23	23.28	22.96	20.02	21.11	23.23
Equivalent Evaporation from and at 212° F. per pound of dry coal, lbs.......	7.24	7.05	7.02	7.11	7.43	7.50
Engine Performance, Dry coal per I. H. P. hour, lbs............	3.91	4.01	4.03	3.98	3.80	3.77
El'ctric'l Perf'rmance Dry coal per Electrical Horse Power lbs..............................	4.82	4.95	4.97	4.91	4.70	4.66
Dry coal per kilowatt hour, lbs...................	6.50	6.66	6.70	6.61	6.32	6.28

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*Bulletin No. 261 p. 75.

In comparing the values of equivalent evaporation given in Table IX with those obtained in every day practice in Iowa power plants, the latter will appear to disadvantage even when intelligent stoking is practiced. It should be remembered that the boilers in the St. Louis tests worked at or near rated capacity at all times and were handled by firemen trained for the tests. The usual conditions of boiler service are variable load and low average load. On page 12 of this bulletin is presented a test of two boilers using Iowa coal under adverse conditions as to load but intelligently fired. At 39 per cent of rated capacity the equivalent evaporation is 5.65 lbs. from and at 212° per lb. of dry coal.

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*Compiled from Bulletin No. 261, p. 81.

**Discrepancies between the analyses here given and those of Table II are due to the fact that special analyses were made for each boiler test from the coal *then* used.—Eds.

Part II

*PRODUCER GAS TESTS BY THE U. S. GEOLOGICAL SURVEY.

Eighteen samples of coal from Alabama, Colorado, Illinois, Indiana, Indian Territory, Kentucky, Missouri, North Dakota, Texas and West Virginia were used to generate gas in a No. 7 Taylor pressure gas producer manufactured by R. D. Wood & Co., Philadelphia, and the gas therefrom was utilized in the cylinders of a three-cylinder vertical Westinghouse gas engine belted to a six-pole 175 k. w. D. C. generator loaded with water rheostats.

Table X shows comparative results of steam and producer gas tests of the samples subjected to these tests. Only fourteen tests are here quoted. Leakage of the producer hopper in the others gave unreliable results.

**TABLE X.

Samples	B. T. U. per lb. dry coal		Average Kilowatts at Switchboard		Dry Coal per Kilowatt hour		Rates of
	Steam Plant	Producer Plant	Steam Plant	Producer Plant	Steam Plant	Producer Plant	Eco-n'my
Alabama No. 2	12,555	13,365	158	148	5.50	2.21	2.48
Colorado No. 1	12,577	12,245	115	148	6.51	2.30	2.83
Illinois No. 3	12,857	13,041	147	148	5.85	2.41	2.43
" No. 4	12,459	12,834	145	148	6.47	2.37	2.73
Indiana No. 1	13,377	13,037	163	148	5.56	2.60	2.14
" No· 2	12,452	12,953	142	149	5.85	2.08	2.81
Indian Ter. No. 1	12,834	13,455	143	152	5.44	2.46	2.21
Kentucky No. 3	13,036	13,226	155	148	5.68	2.57	2.21
***Missouri No. 2	11,500	11,882	152	148	6.62	2.30	2.88
West Va. No. 1	14,198	14,396	146	148	5.25	2.12	2.48
" " No. 4	14,002	14,202	157	148	4.87	1.74	2.80
" " No. 9	14,616	14,580	154	149	4.66	2.14	2.18
" " No. 12	15,170	14,825	151	148	4.75	2.02	2.35
Wyoming No. 2	10,897	10,656	135	149	7.94	2.78	2.85
Averages	13,037	13,192			5.71	2.29	2.49

The favorable showing of the producer gas engine in these tests is significant. While it is true that the steam engine used was a simple non-condensing engine having a "water-rate" of 23.6

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*Bulletin No. 261 p. 85.
**Compiled from p. 119, Bulletin No. 261.
***Macon County.

pounds, it is also true that the gas engine in the large sizes is still in an experimental stage especially in those features of its design and operation which affect its utility in plants where only ordinary skill can be expected to be exercised.

Of the coals listed in Table X, Missouri No. 2 resembles most closely the Iowa coals—its principal properties being

Moisture	11.60
Carbon, Volatile	35.28
Carbon, Fixed	38.28
Ash	14.84
Sulphur	4.56
Calorific Value	11500 to 11882

and the average of the Iowa coals in Table II being

Moisture	13.16
Carbon, Volatile	33.36
Carbon, Fixed	39.69
Ash	13.76
Sulphur	4.65
Calorific Value	10019 to 11027

"The high percentage of sulphur** in the coal (Missouri No. "2) did not add to its value as a producer fuel, but the writer's "opinion is that it can be used to advantage in producers by increasing the facilities for purifying."

The lack of correspondence between the relative values of the several coals in Table X for steam tests and for producer tests indicates that a given producer may be better adapted for handling a wide variety of coals than is a given boiler furnace.

Table X also shows that for these tests and conditions the percentage saving in fuel of the producer over the steam is greater for the poorer coals. Compare Wyoming No. 2, Missouri No. 2, Colorado No. 1 and Illinois No. 4 with West Virginia Nos. 1, 9 and 12, Indiana No. 1 and Illinois No. 3. West Virgimia No. 4 seems to be an exception.

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*Bulletin No. 261, p. 102.

**The high sulphur increases the risk of corrosion of the engine cylinder unless the purification of the producer gas is very thoroughly provided for by the designer and looked after by the operator of the plant.

This is a reasonable view also because the volatile constituents of the coal in the producer escape only through the engine cylinder, in which their combustion is quite completely effected, whereas in steam generation with various fuels under a given boiler, various proportions of volatile constituents escape to the chimney unburned.

It should be noted, however, that the producer gas engine power plant is relatively much more complicated than the steam engine plant and requires for its successful operation more technical knowledge and skill, and these conditions should be had in mind by plant owners and managers, especially by those do not believe in paying wages sufficient to secure high grade service.

Part III

TEST OF HARD COAL PRODUCER GAS ENGINE AT ALGONA, IOWA.

At the City Electric Light Plant, Algona, Iowa, there has been installed a three cylinder vertical Fairbanks-Morse gas engine, using gas generated from anthracite pea coal in a suction producer, the latter being also manufactured and installed by the Fairbanks-Morse Co. The unit is rated at 150 brake horse-power at 250 revolutions per minute, and was guaranteed to give one brake horse-power hour for 1½ lbs. of anthracite pea-coal for all loads above 75 brake horse-power.

Through the courtesy and assistance of the city and of the builders, brake tests of this engine have been made by the College. Table XI shows the results.

TABLE XI.

Revs. per Min.	Brake Load Horse Power	Lbs. Coal (as fired) per Brake Horse Power per hour	Cost per Brake Horse Power Hour at $6.00 per ton
250	40.1	1.511	$0.00453
250	82.7	1.157	.00347
250	156.9	0.999	.00299

Two tests were also made on this engine under service conditions viz: belted to a 75 K. W. alternating current generator. In addition to the lighting load, electrically driven pumping machinery can be operated from this generator.

Fig. 1 shows the load curve (A) during a service run with lighting load only and the load curve (B) for the combined load, the usual operating conditions.

At $6.00 per ton the cost of fuel per kilo-watt hour at the switch-board for the load A was $0.01207 including fuel for banking and starting, and for the load B was $0.00639 including also the stand-by losses.

Soft coal from Illinois, which was used for a Corliss engine unit in the same plant cost $3.40 per ton.

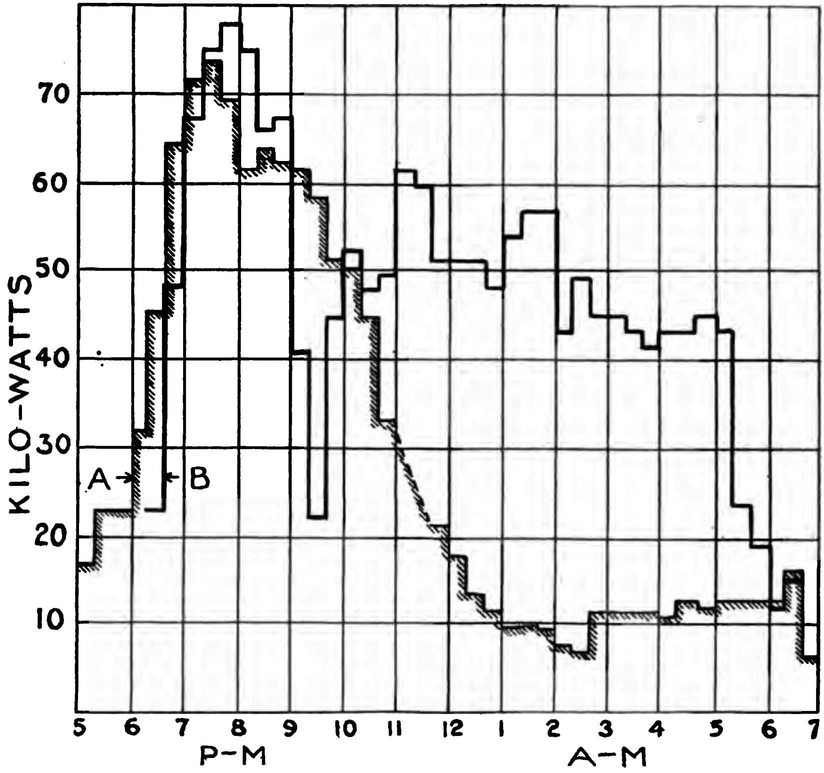


FIG. I.

A.—Load Curve of Algona Producer Gas Engine, Mar. 15-16, '06.
Anthracite Pea Coal per K. W. H. = 4.10 lbs.
Fuel Cost per K. W. H. = $0.0123.

B.—Load Curve of Algona Producer Gas Engine, Mar. 16-17, '06.
Anthracite Pea Coal per K. W. H. = 2.23.
Fuel Cost per K. W. H. = $0.00699.

Part IV

TESTS OF MECHANICAL STOKERS WITH IOWA COAL

The two things to be desired by all steam plant owners and managers in the use of coal for steam generation are economy and smokelessness.

Intelligent hand-firing is the best agency yet known for securing these. But unfortunately intelligent firemen are not easily obtained or retained and in large plants their cost *may* be prohibitive as compared with mechanical stokers.

Consequently the mechanical stoker has been rapidly developed in many localities and has met the demands of economy and sometimes also of smokelessness.

Owing to the difficulties presented by Iowa coals, the tendency among plant owners to be conservative in initial costs, and to the lack of data of actual performance of mechanical stokers in Iowa, their introduction into this field has been slow.

So far as known to the writers the principal installations of mechanical stokers in Iowa are as follows:

Des Moines—Iowa State Capital, Des Moines Street Railway, Des Moines Edison Company.

Cedar Rapids—Cedar Rapids Electric Light Company, American Cereal Mills.

Ottumwa—John Morrell & Co., Ottumwa Electric Light & Traction Co.

Dubuque—Union Electric Co.

In the spring of 1905, Prof. W. H. Meeker, assisted by Messrs Bullock, Ricker, Scott and Smith of the Senior class in Mechanical Engineering at the Iowa State College, made four tests of a Stirling boiler equipped with a Murphy Smokeless Furnace, the principal general results of which are exhibited in columns 1 to 4 of Table XII, and three tests of a Stirling boiler equipped with a Chain Grate Stoker, the principal general results of which are exhibited in columns 5 to 7 of Table XII.

All usual and necessary precautions were taken in the preparations for and the conduct of the tests to secure reliable data. Anticipating the criticism that most of the tests were of short dur-

TABLE XII

GENERAL RESULTS OF SEVEN TESTS ON MECHANICAL STOKERS USING IOWA COAL

Test No.	1	2	3	4	5	6	7
Name of Stoker	Murphy Smokeless Furnace				Chain Grate		
Description of Fuel, All Freshly Mined	Smoky Hollow Steam Coal	Anchor Steam Coal	F. & W. Roseland Steam Coal	F. & W. Avery Steam Coal	Colfax Steam Coal	Flint Brick Coal	Norwood ville Steam Coal
ANALYSIS OF FUEL							
Carbon, Fixed ... Percent	37.8	41.7	41.25	39.50	41.50	39.50	38.40
Volatile ... Percent	35.4	33.0	30.77	34.42	30.80	30.19	32.35
Moisture ... Percent	10.8	10.0	11.23	11.08	11.50	13.06	14.25
Ash ... Percent	16.	15.3	15.75	15.00	16.20	16.25	15.00
Calorific Value per lb. of Fuel ... B. T. U.	9,719	9,963	8,594	9,655	10,742	9,952	10,479
Duration of Test ... Hours	5-½	9	5	5	5	5	7
Total Moist Coal Consumed ... lbs.	14,932	20,558	8,400	12,975	13,801	12,942	20,714
Total Ash and Refuse ... lbs.	2,903	4,110	1,700	2,477	2,668	2,525	3,570
" " " " ... Percent	19.4	19.9	20.2	19.0	19.3	19.5	17.2
Dry Coal per Square Foot of Grate per Hour ... lbs.	40.36	34.26	24.88	38.46	25.7	23.7	26.7
Equivalent Evaporation from and at 212° F. per lb. of Dry Coal ... lbs	5.28	6.12	5.66	5.20	5.45	5.95	6.02
Horse Power Developed by Boiler ...	371.00	365.2	244.6	348.1	385.7	389.8	442.4
Percentage of Builder's Rating ...	110	109	73	103	87	88	100
Excess of Air Supplied ... Percent		73	91	95	131	127	91
Cost of Coal per Ton of 2,000 lbs. ...	$1.324	$1.444	$1.664	$1.214	$1.07	$1.20	$1.27
Cost of Coal per 1,000 lbs. of Steam from and at 212° F.	$.14	$0.13	$0.165	$0.131	$0.111	$0.115	$0.123
Boiler Efficiency ... Percent	55	63.2	66.7	55	51.02	59.9	57.3
Draught in Furnace ... Inches	0.19	0.202	0.14	0.26	0.202	0.195	0.195
Flue Gas Temperatures ... F.	637.4	678	652.6	686.4	598.1	626.2	622
Average CO^2 in Flue Gas ... Percent		9.2	7.7	8.1	7.1	7.6	7.8

ation, the writers would say that the mechanical feeding of the fuel and the possibility of maintaining approximately constant load on the boilers during the tests are considered sufficient justification.

The coals used in the several tests were

Test No.	1.	Smoky Hollow Steam Coal.
"	2.	Anchor Steam Coal.
"	3.	Roseland Steam Coal.
"	4.	Avery Steam Coal.
"	5	Colfax Steam Coal.
"	6.	Flint Brick Steam Coal.
"	7.	Norwoodville Steam Coal.

The average of the results of the four tests on the Murphy and of the three tests on the chain grate furnace are given in Table XIII.

TABLE XIII.

Name of Stoker Number of Tests	Murphy 4	Chain Grate 3
Ratio of Heating Surface to Grate Surface	56	46.7
Average Steam Pressure	99.9	137.4
Draught in Furnace	0.198 ins.	0.197 ins.
Temperature, Flue Gases	664° F.	615° F.
ANALYSIS OF FUEL		
Carbon, Fixed	40.0 percent	39.8 percent
Volatile	33.5 "	31.1 "
Moisture	11.0 "	12.9 "
Ash	15.5 "	15.8 "
Calorific Value, 1 lb. Dry Fuel	9483 B. T. U.	10391 B. T. U.
CO_2 in Flue Gas	8.3 percent	7.5 percent
Equivalent Evaporation from and at 212° F. per lb. Dry Coal	5.56 lbs.	5.80 lbs.
Dry Cool per sq. ft. Grate Surface per Hour	34.4 "	25.3 "
Total Ash and Refuse	19.6 percent	18.6 percent
Boiler Efficiency	59.9 "	56.1 "

Part V

MISCELLANEOUS BOILER TESTS

Through the kindness of Professors Breckenridge and Randall of the University of Illinois, the writers have access to the results of 77 boiler tests made by them or under their direction on Illinois coals in hand fired and mechanical fired boiler glants. In Table XIV is presented a summary of these tests grouped according to types of grates and furnaces employed.

TABLE XIV.

Type of Grate or Furnace	Hand Fired			Mechanical Stoker		
				Murphy	Roney	chain gr'te
Type of Boiler........	Hor. Ret. Tubular	Water Tube		National water tube	B. & W. water tube	B. & W. water tube
		Heine	B. & W.			
Number of tests....	23	5	29	9	2	18
Ratio of Heating Surface to Grate Surface..................	28.4 to 31.8	50.1	42.4 to 53.1	41.8	46	53.1
Steam Pressure......	64.6	84.4	67.8 to 108.5	93.2 to 108.1	117.8 to 119.4	103 to 120.1
Temperature of Flue Gas..	520 to 622	554 to 637	420 to 490	481 to 515	559 to 624	482 to 534
C O^2 in Flue Gas..		7.62				
Equivelent Evaporation from and at 212° per lb. dry coal..	5.92 to 6.48	7.21 to 8.04	4.20 to 5.76	6.04 to 6.28	6.43 to 7.75	6.13 to 7.49

Following is outline report of test of boilers at Marshalltown, Iowa. The boilers furnished steam for pumping through 24 hours against direct pressure and for all night street lighting service:

RESULTS OF MARSHALLTOWN TEST.

Feb. 9-10, 1906, by G. W. Bissell and Senior M. E.'s of two (2) H. R. T. boilers built by Shorthill Mfg. Co. located in City Water Works Pumping Station, Marshalltown. Duration 24 hours.

DIMENSIONS AND PROPORTIONS (EACH BOILER)

Shell—68 in. diam., 16 ft. long.
Tubes—66 3½ in. diam., area through tubes, 3.82 sq. ft.
Grate—5 feet square. Area 25 sq. ft.
Heating Surface—1108 sq. ft.
Ratios—Grate surface to heating surface, 1 to 44.
Least draught area to grate surface, 1 to 6.5.
Least draught area to heating surface, 1 to 286.
Chimney—Height, 100 ft; 5 ft. square inside (for 3 boilers.)

FUEL.

Name and Source—Jasper County Steam Coal.
Moisture—4.8%.
Heat Units per Pound Day Coal—8100 by Calorimeter.

AVERAGES OF DATA AND RESULTS (BOTH BOILERS)—PRESSURES.

Steam in Boiler by Guage—95 lbs. per sq. in.
Draught in Breeching—0.499 ins. water.

TEMPERATURES.

External Air—6.6° F.
Boiler Room—67.4° F.
Steam—335° F.
Feed Water from Hot Well—86.5° F.
Feed Water at Boiler—160.8° F.
Steam in Calorimeter—236.3° F.
Flue-gases in Breeching—317.8° F.

FLUE GAS ANALYSIS.

CO_2—6.7% by volume.

FUEL.

Total Moist Coal—13473 lbs.
Total Dry Coal—12626 lbs.
Total Ash and Refuse—2382 lbs.
Dry Coal per Hour—529.9 lbs.
Dry Coal per Hour per Sq. Ft. of Grate—10.6 lbs.
Heat Units Supplied per Hour—4,292,190.

QUALITY OF STEAM.

Moisture—2.94%.

EVAPORATION.

Total Water Supplied—67891.3 lbs.

Total Water Evaporated to Dry Steam—65854.5 lbs.

Factor of Evaporation—1.092.

Total Equivalent Evaporation from and at 212° F.—71913.1 pounds.

Per Hour—Water Supplied—2826.8 lbs.

Dry Steam—2743.9 lbs.

From and at 212°—2996.3 lbs.

From and at 212° per sq. ft. Heating Surface—1.35 lbs.

Heat Units Absorbed by Dry Steam—2,983,526.

Heat Units Absorbed by Water per Pound Dry Coal 5455.

EVAPORATIVE PERFORMANCE.

Actual Water per Pound of Coal—4.88.

Equivalent Evaporation from and at 212° per 1 lb Dry Coal—5.65

EFFICIENCY.

Boiler and grate—67.3%.

HORSE POWER.

Developed—86.8=39.4% of Rating.

COMMERCIAL EFFICIENCY.

Cost of Fuel in Boiler Room—$1.80 per ton.

Cost of Fuel for 1000 lbs. Steam, Actual Conditions—$0.18 2-5.

Cost of Fuel for 1000 lbs. from and at 212°—$0.16 4-5.

Part VI

CALORIMETER TESTS

In Table XVI are presented the heating values and in some cases other information obtained from samples of Iowa coal gathered from boiler rooms and car lots as delivered to consumers. The Parr Standard Coal Calorimeter was used in the calorimetric determinations.

TABLE XVI.

Designation	County	Grade	B. T. U per lb. Dry Coal	Proximate Analysis				
				Carbon		Ash	Moist- ure	Sul- phur
				Vola- tile	Fixed			
Smoky Hollow		Steam	9719	35.4	37.8	16.0	10.8	
Anchor		"	9963	33.0	41.7	15.3	10.0	
Anchor		Lump	11027	30.7	45.0	16.0	8.2	5.03
Roseland		Steam	8594	30.7	41.2	15.7	11.2	
Avery		"	9655	34.4	39.5	15.0	11.0	
Colfax	Jasper	"	10742	30.8	41.5	16.2	11.5	
Flint Brick	Polk	"	9952	30.1	39.5	16.2	13.0	
Norwoodville	"	"	10479	32.3	38.4	15.0	14.2	
Mammoth Vein	Marion	Lump	10019	33.1	37.4	15.2	14.2	4.66
Gibson Coal Mining Co.	Polk	"	10244	36.9	35.1	14.0	13.8	6.15
Centerville Blk	Appa- noose	"	10723	35.5	39.3	10.9	14.0	4.26
Inland Fuel Co.	Lucas	"	10242	30.4	41.4	12.6	15.3	3.19
*Johnson, 9	Boone	Slack	7363	22.1	26.1	40.1	11.7	
" 5	"	Lump	11412	27.7	41.6	15.2	15.3	
Rogers, 3	"	Slack	7463	20.4	26.1	41.2	12.0	
Heaps & Crowe 1	"	Lump	9905	27.8	32.9	26.0	13.3	
Heaps & Crowe 4	"	Slack	7588	14.8	31.2	42.0	12.0	
Marquisville	Polk	Nut	11136	30.0	43.5	20.6	5.8	

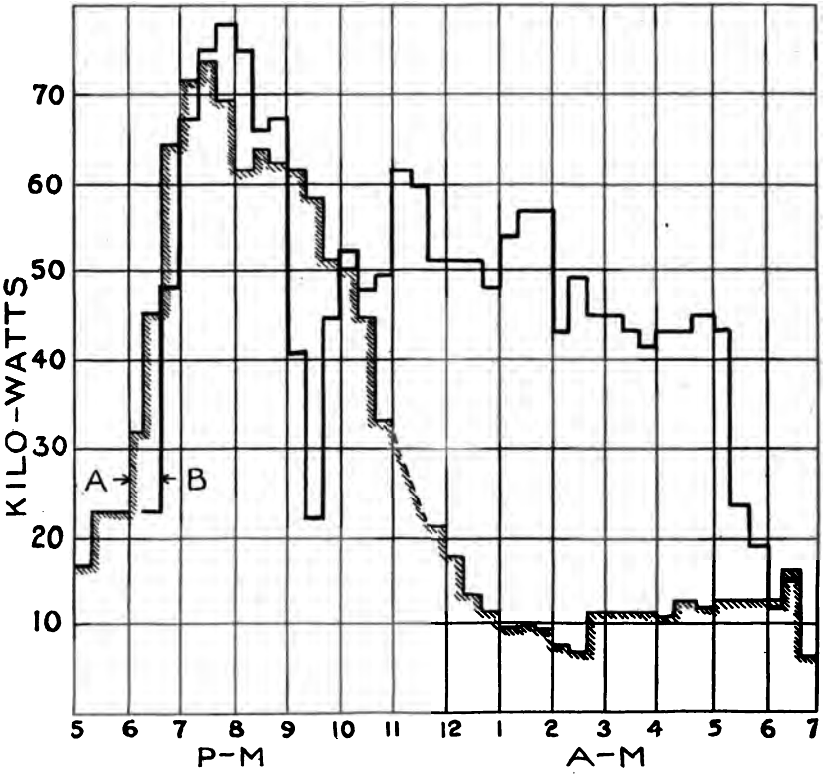
*The number of cars of each kind from which samples were obtained is indicated by the numerals.

Part VII

POWER STATION LOAD CURVES AND LOAD FACTORS

The load factor of a power station is the ratio of the average output to the rated capacity of the plant. Attention is called to Fig, 1, here reproduced, showing the load curves for the power station at Algona under two different conditions. The method of computing the load factor is here exemplified by the use of data from this plant, taking first the operating conditions represented by the load curve A for lighting service only. The gas engine, which was rated at 150 brake horse power was direct belted to a 75 kilo-watt 2-phase generator, but should carry a 90 K. W. machine and the latter is used in the following discussion:

FIG. I.



The total kilowatt-hours of output for the run was 409 and the length of the run was 14 hours. If the full capacity of the 90 K. W. generator for the 14 hours had been delivered the output would have been 1260 kilowatt hours. The apparent load factor is the ratio of actual to capacity output, 409÷1260=32 per cent but the real load factor is based upon the 24-hour capacity and is 409÷(90x24)=18 per cent.

Similarly for load curve B, for which the output was 589 kilowatt hours for 12 hours, the load factors were
apparent load factor=54 per cent
real " " =27 per cent

It is worthy of note that the increase of load factor shows in the fuel cost per K. W. hour, which for A was $0.01207 and for B was $0.00639. corresponding to 4.02 and 2.13 pounds of coal per K. W. hour respectively. (See page 13) The above exhibit relates to two daily runs.

Table XVI shows the monthly kilo-watt hour output of a plant furnishing public and commercial lights, general power and street railway service. The data covers one year. The total capacity of the plant is 920 kilo-watts. The annual load factor on the basis of 24 hours of service is as follows:*

For the railway service.......... ...31.0 per cent
For the lighting and power..............17.0 per cent
For the whole plant.......... ....18.5 per cent

The load factor month by month for the same plant is as follows:

TABLE XVI.

January,	total	out	put,	141,784	K. W. H.,	Load	Factor,	21.0	per	cent.
February,	"	"	"	116,010	"	"	"	17.5	"	"
March,	"	"	"	120,308	"	"	"	18.1	"	"
April,	"	"	"	109,194	"	"	"	16.2	"	"
May,	"	"	"	105,826	"	"	"	16.0	"	"
June,	"	"	"	98,730	"	"	"	14.8	"	"
July,	"	"	"	107,854	"	"	"	16.3	"	"

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*The data for this analysis are not given in Table XVI.

August,	total out put,	113,156	K. W. H.,	Load Factor,	17.0	per cent.
September,	" " "	117,470	"	" "	17.6	" "
October,	" " "	144,880	"	" "	21.8	" "
November,	" " "	152,448	"	" "	23.0	" "
December,	" " "	172,530	"	" "	26.0	" "
For Year,	" " "	1,500,530	"	" "	18.5	" "

Table XVII presents the relations between fuel and cost per kilo-watt hour and load factor for a station of 2000 K. W. capacity, coal costing $2.40 per ton.*

TABLE XVII.

Load Factor per cent	Cut of Fuel per K.W.H.	Per Cent of Cost for Load Factor of 70 per cent
10	$0.012	315
15	.01	264
20	.008	210
25	.007	183
30	.0064	169
35	.006	158
40	.0052	137
45	.005	132
50	.0048	126
60	.0044	116
70	.0038	100

In the last column of Table XVII the unit cost for 70 per cent load factor is taken as unity. This assumption would nearly represent actual condition where a reserve unit is installed.

In addition to the importance of the load factor as it affects operation cost is its effect upon total cost. The fixed charges are a larger proportion of the total expense in a plant with a low load factor than in the same plant with a high load factor.

*Adapted from Engineering and Electric Traction Pocket Book—Dawson, p. 1287.

Engineering Experiment Station

IOWA STATE COLLEGE

Ames, Iowa

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BULLETIN
Vol III No. 4

Electrical Engineering Section

Tests of Incandescent Lamps

FEBRUARY 1906

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Introduction

Comparatively few of those who have to deal with the incandescent lamp have an adequate conception of the real significance of lamp testing or a definite knowledge of the methods employed in testing such devices. It has therefore seemed advisable to preface the subject matter of this bulletin by a general discussion on the subject of lamp testing and lamp rating, and a brief outline of the various tests to which incandescent lamps are commonly subjected.

So far as the results of the tests herein outlined are concerned, the topic of mean spherical rating has not been discussed to any extent. It is proposed to extend this testing work and in a later bulletin this and other related topics will be taken up in detail.

Iowa State College L. B. SPINNEY.
February 1906

Tests of Incandescent Lamps

It is a well known fact that a very large part of the data upon which it is customary to rely for a knowledge of the behavior of the incandescent lamp under varying conditions of voltage and efficiency, has originated in the factory, and that comparatively few lamp tests of a general nature have been made in a disinterested way. Those who care to trouble themselves with investigations of the nature above referred to, are, in general, compelled to send their lamps to a great distance to some standardizing bureau or to provide themselves with the necessary apparatus for making the tests. The consequence is that very little work of this kind is done since either procedure is often times out of the question because of the expense involved.

At the same time the question is of sufficient importance to warrant the most extended and painstaking investigation.

From the standpoint of the plant owner or operator it is a matter of importance since it is his business to furnish proper illumination for the premises of his customers at a cost consistent with the service rendered. From the standpoint of the lamp user it is equally important because his interests demand the efficient illumination of his premises at a reasonable cost.

In planning the average electric light plant attention is justly given to the efficiency of the boiler room installation because in this equipment even under best conditions but a low efficiency is obtained. It is also insisted upon that the electric machinery be of standard manufacture and efficient in every respect, although relatively speaking, even in the less satisfactory forms of apparatus of this character, a high efficiency is secured. On the other hand, the efficiency of the lamp used receives little or no attention. At first thought the lamp efficiency may appear of equal importance with that of the boiler or dynamo efficiency, for the energy available in the lighting system must pass through each of these devices, and the efficiency of each part of the plant appears as a factor in the general result. When it is remembered, however, that the commodity which is bought and sold, which the plant operator agrees to furnish and the lamp user agrees to pay for is light,—it must be evident that the lamp really takes on a superior importance in this connection. However much energy may be developed in the boiler and transformed in the engine and dynamo it is of course the lamp which determines the availability of that energy in the useful form.

This point being conceded the importance of photometric lamp tests is at once established for the only means of determining lamp efficiencies is by the use of the photometer. The efficiency of a lamp, as in the case of any other device for the transformation of energy, is given by the ratio of the useful output to the total input. The useful output in the form of light is measured in candle power. The input in the form of electric energy is measured in watts. The efficiency would therefore be logically expressed in candle power per watt. Custom has, however, established the use of the inverted efficiency and a lamp is commonly described as taking so many watts per candle power.

It cannot be said that high efficiency lamps should always be used. Indeed, it may be shown that under certain circumstances best practice would point to the use of lamps of low efficiency. In any given case there is always a number of things to be taken into account in the determination of that efficiency which is best adapted It will be understood that in those cases in which the cost of power, for any reason, is excessive, much is to be gained by the use of high efficiency lamps in spite of the greater cost of renewals due to the shorter life of such lamps. On the other hand, for plants operated by water power in which increased load does not involve increased operating expense, it is conceivable that a lamp of lower efficiency might be desirable, especially if regarded solely from the standpoint of economy in the operation of the plant.

It is not proposed to enter at this time upon a detailed discussion of the various factors which must be considered in determining that lamp efficiency which is best adapted to given conditions or any particular case. Suffice it to say that for each set of conditions there is one efficiency which is better adapted than any other. Having reached a conclusion in this matter it behooves the plant manager to make sure that lamps of the desired efficiency are placed in operation.

One does not need to make a very comprehensive series of lamp tests in order to convince himself that the various lamps on the market are widely different in their characteristics or that there is often an appreciable difference even in lamps of the same manufacture. Of course, it goes without saying that it is impossible to manufacture lamps in large numbers that are exactly alike and a certain variation is to be looked for and accepted but this fact hardly makes the test for general averages less important or instructive nor does it in any wise warrant the manufacturer in relaxing his vigilance in the production of a lamp as constant in its properties as may within practical limits be secured.

It would seem that common business considerations would convince the station manager that he should have definite knowledge of the performance of his lamps, for it would be folly to make a painstaking determination of the most suitable lamp efficiency and then fail to make sure that lamps of such efficiency were employed. This statement is to be taken not as referring to the large users of lamps alone, but to any and all upon whom the management of a light plant may fall.

Another consideration in this connection is the decreased efficiency with the life of the lamp. The practice, not at all uncommon in smaller plants, of leaving lamps in circuit until they are burned out, is of course to be condemned. Aside from the consideration of efficiency there is that of the annoyance and dissatisfaction to the customer which is always attendant upon the use of old lamps. It is unquestionably true that the greater part of the dissatisfaction among the users of the electric light is to be attributed to dim lamps.

It is now very generally agreed that a lamp should be replaced when its candle power has fallen to eighty percent of its initial value. In order that lamps may not be kept in circuit beyond the limit of useful life it is necessary either to keep a record of all lamps as to date of renewals and the hours per day burned and rely upon a knowledge of the average behavior of the lamps used for determining the proper dates for renewals or to make photometric tests of all lamps from time to time rejecting those which have fallen in candle power below a certain value. Perhaps a better method than either of those given is a combination of the two.

THE RATING OF INCANDESCENT LAMPS

The following discussion of the subject of The Rating of Incandescent Lamps is taken from a paper by the author of this bulletin read before the Iowa Electrical Association in April 1905.

"The candlepower of an incandescent lamp, as commonly rated, is a specification of its average illuminating effect in a horizontal direction, the axis of the lamp being vertical, and its efficiency is given in terms of this horizontal illuminating effect and the total power consumed.

While it is true that most lamp makers and lamp users accept this method as a convenient means of rating an incandescent lamp, it must be admitted that the chief reason for adopting it has been the facility with which such rating is obtained, and that, aside from this reason, there is very little argument in favor of its use. It has been pointed out again and again that this method of rating an incandescent lamp or other source of light is illogical; but, because no convenient means has been suggested for obtaining the average illuminating effect in all directions—that is to say, the mean spherical candle power—those who have to do with lamp testing and lamp rating have clung to the horizontal candle power method as the only practical one for comparing the illuminating effects of various lamps.

It would seem that a very simple consideration of the question would convince one that the only logical rating of any source of illumination is in terms of total illuminating effect and the total power consumed. It must be admitted that in some cases we are particularly concerned with the illumination produced in a given direction and that for these special purposes the consideration of the mean spherical candle power of the source is not so important. Nevertheless, in determining what return is made for that which is put into a given lamp, it is necessary to take account of the total illuminating effect secured. This is true for lamps used for these special purposes, as well as for others. And in order that we may be able to make efficiency comparisons between lamps of this sort, we must know, in addition to the illumination secured in the desired direction, the mean spherical candlepower. It will be evident that of two lamps, used thus for any special purpose and giving equal illuminating effects in the desired direction, that one is the more efficient which for a given consumption of power gives the greater mean spherical illumination. It will seldom happen in practice that after the illumination in the desired direction is assured that any outside illuminating effect will be found objectionable. It is readily conceived that in case of two so-called "downward candlepower" lamps the downward effects may be equal, while the mean spherical effects may be more or less widely different.

It will thus be seen that even though we are discussing lamps which are designed to give a certain candlepower in some one direction that the question of mean spherical candlepower is still of importance. It would be absurd to attempt to compare heating appliances in terms of the total heat radiated in a horizontal direction. Is it not really quite as illogical to attempt to rate light sources in the same way?

It is true the analogy is an imperfect one, since the convection currents about any heating appliance make the vertical effect of relatively greater importance in such a device than it would be in the case of an incandescent lamp of any of the ordinary forms. At the same time in the average problem in illumination the vertical effect of the radiant is important and the analogy is useful, in that it emphasizes the need of taking this effect into consideration.

The illumination at any point in a room, for example, is determined in part by the light which falls upon it directly from the source and in part by the diffusely reflected light which reaches it from other parts of the room. In the production of this secondary effect the ceiling and upper parts of the walls play an important part, especially so since these are ordinarily the lighter parts of the room. This diffusely reflected light is again determined largely by the illumination which the ceiling and upper walls receive directly. It is absurd, therefore, to ignore this effect in an attempt to predetermine the illuminating effect of an incandescent lamp for any point in the room, even for those points which lie beneath or in a horizontal direction from the source.

Again, the term efficiency, as applied to the ratio of the horizontal illumination and the total power supplied to the lamp, is meaningless. The efficiency of any device for the transformation of energy is defined as the ratio of the total energy transformed, to the total energy supplied to the transforming device. And for purposes of specification there neither is nor can be any other conception of its meaning.

It has been urged, as one of the chief objections to the method of mean spherical candlepower rating, that the measurement of the mean spherical candlepower of any source of illumination, and particularly of a source for which the distribution is far from uniform, involves great difficulties and tedious labor. It has been a weighty argument and has perhaps done more than any other one thing to establish the rating of lamps in terms of their mean horizontal candlepowers, which are much more easily obtained.

It must be admitted that, in the case of commercial ratings, this argument has been a difficult one to meet. In recent times, however, methods have been suggested and apparatus has been devised by means of which the mean spherical candlepower of a lamp is quite conveniently determined.

Therefore, the argument just referred to in favor of the mean horizontal candlepower method no longer carries the same weight as formerly, and it seems reasonable to expect that in the near future more and more attention will be given to mean spherical candlepower measurements. It is even conceivable that before long incandescent lamps will be rated in terms of their mean spherical candlepowers rather than in terms of the average horizontal candlepower, as is customary at the present time.

THE VARIOUS TESTS TO WHICH INCANDESCENT LAMPS ARE SUBJECTED

It will appear from the facts set forth above and from other considerations, which will occur to anyone at all familiar with the use of the incandescent lamp, that there are several distinctly different tests which may be made upon a lamp. The relative importance of these several tests differs according to the objects of the investigations. For example, one lamp user may wish to compare various brands of lamps for the purpose of determining which has the longest useful life. In such a test the question of "selection" might very well be of minor importance. In another case the kind of lamp to be used may have been decided upon and the user may wish a test for "selection" to protect himself from the careless or unscrupulous dealer who might supply him with lamps of higher or lower efficiency than those contracted for. In such cases the question of "life" would propably receive little or no attention.

The several tests to which incandescent lamps are commonly subjected are as follows:

Initial candle-power and wattage tests.
Life tests.
Candle-power-hour tests.
Vacuum tests.

In a comprehensive study of a lamp there would also be included an inspection to determine the presence or absence of mechanical defects, such as loose and imperfect bases, "spotted" filaments, lack of symmetry in bulb or filament, discoloration and flaws in bulb, discolored carbons, etc.

INITIAL CANDLE POWER AND WATTAGE TEST

This test is usually made for the purpose of determining the "selection" of lamps, that is, the care with which they have been selected as to voltage to give the candle power and efficiency at which they are rated Generally speaking, lamps of any one kind which stand this test will give uniform results as to the total light produced and total power consumed.

The results of this test are usually represented in the so-called "target" diagram. In this diagram the results of an individual lamp test are represented by a dot. The distance of this dot from the bottom of the diagram measures the candle power of the lamp while the total power (measured in watts) consumed by the lamp is indicated by the distance of the dot from the left hand edge of the diagram.

It is a simple matter to increase or decrease the efficiency of a lamp by raising or lowering the voltage which is brought to bear upon its terminals. This change in efficiency, however, is accompanied by a change in the candle power of the lamp, so that once the efficiency is specified the candle power is fixed and vice versa. It will therefore be evident that not only is skillful manufacture necessary, but careful selection as well, in order that a given number of lamps may operate at a prescribed voltage and efficiency.

It is found to be practically impossible to manufacture large numbers of lamps which are identical in their candle powers and efficiencies. A certain maximum departure from the nominal rating is therefore allowed It is generally agreed that properly rated lamps should measure within 1½ candle power above and 1½ candle power below the rated candle power and within 6% above and 6% below the rated wattage. The "target" in the diagram encloses an area corresponding to these allowable variations in candle power and wattage. In a diagram for properly rated or carefully selected lamps all dots should therefore fall within the target.

LIFE TEST

The normal incandescent lamp when operated at its rated voltage at first increases in candle power. This increase continues however for a relatively short time only, after which, the candle power decreases more or less rapidly to the end of the useful life of the lamp. It is not considered good practice to keep a lamp in service after the candle power has fallen below 80% of its initial value. It is on this basis that comparative life tests are made. The life of a lamp is thus the length of time it will operate before reaching this 80% value.

CANDLE-POWER-HOUR TEST

For this test the lamp is operated at its rated voltage and its candle power measured from time to time throughout its useful life. The average candle power, determined from these measurements, is multiplied by the life of the lamp. This product gives the useful output of the lamp in candle power hours. The result is used as a basis for comparative tests.

VACUUM TEST

The life of an incandescent lamp depends to a large extent upon the care with which the lamp has been exhausted, i. e., upon the vacuum within the bulb. If any appreciable amount of air or gaseous material is left within the lamp, the bulb will blacken within a short time and the candle power of the lamp will fall off rapidly. Lamps of this nature are technically known as "slumpers." The test which is usually made for determining the vacuum is to hold the bulb in the hand and present the base to one of the terminals of a small induction coil. A low vacuum is indicated by a blue glow within the bulb. If the vacuum is good the glow is but faint and flashes in a fitful manner which is characteristic. The temperature of the bulb when the lamp is in circuit is also a rough indication of the quality of the vacuum. If the bulb is so hot as to burn the hand the vacuum is low. If the vacuum is good the bulb may be held in the hand with but little discomfort. Another test which may be used is to give the lamp a sudden jar. If the vacuum is low the vibrating filament will quickly come to rest. In a good vacuum the filament will continue to vibrate for a longer time.

THE LAMP TESTS MADE AT THE IOWA STATE COLLEGE

Object. The object of these investigations has been to determine the care with which average commercial lamps are manufactured and selected, to compare some of the leading makes of lamps in these respects and to secure unbiased data for the comparison of mean spherical and mean horizontal ratings.

Scope. In these tests something over 1000 lamps representing 32 different makes have been studied. These lamps range in rated candle power from 1 to 50, in rated voltage from 50 to 225 and in rated efficiency from 3.1 to 4 watts per candle power. The majority, however, are rated 16 candle power lamps and excepting in the vacuum tests, these only are included in the tables, diagrams and discussions given herewith.

The lamps included in these tests were secured through the cooperation of 41 different electric light plants or light users, 38 of these being located within the state. The lamps were sent to the Department of Physics and Electrical Engineering in lots of from six to eighty-two lamps, the average number of lamps from each source being seventeen.

The Electrical Section of the Experiment Station offers to test lamps for the citizens of Iowa at a price which but covers the cost of the work and the depreciation of the apparatus used in making the tests. A large number of the tests discussed were made for lamp users who wished to take advantage of this offer and desired the tests for their own purposes and who had no knowledge of the proposed bulletin on lamp testing. All others were made on lamps solicited for the bulletin work. The latter were made free of charge to the sender, transportation charges excepted.

The following list gives the name and location of each source from which lamps were received for these tests together with the number of lamps received from each source. This list includes all 2, 4, 8, 16 and 32 candle power lamps received from these sources:

NAME.	NO. OF LAMPS.
Alline, J. K., Fort Dodge	6
Belle Plaine Electric Light Company	15
Brice Gas and Electric Co., Mason City	18
Burlington Railway and Light Co.	6
Cascade Milling Co., Sioux Falls, S. D.	30
Cedar Rapids and I. C. Rry. & Lt. Co,	24
Cherokee Electric Co.	18
Citizens Light, Heat & Power Co., Eagle Grove	24
City of Ames	30
College. Iowa State College	32
Denison Electric Light and Power Co.	11
Dyersville Lt. & Power Co.	8
Greene Electric Light & Power Co	12
Hampton Electric Lt. & Power Co.	12
Iowa City Electric Light Co.	12
Iowa Falls Electric Light & Power Co.	8
Jefferson Heat, Light, Power and Water Co.	12
Knoxville Electric Co.	10
Manning Electric Light Co.	5
Marengo Electric Light Co.	19
Missouri Valley Electric Light Co.	16
Munns Electric Company, Corning, Iowa	12
Nashua Electric Co.	12
Northwood Electric Light Co.	12
Orange City Electric Light Co.	16
Osceola Electric Light Co.	10
Ottumwa Traction and Light Co.	82
Paris City Water and Light Plant, Paris, Tenn.	32

Pella Electric Light and Power Co.	12
Peoples Light Co., Davenport, Iowa	12
Perry Electric, Power and Heat Co.	24
Red Oak Electric Company	42
Sac City Electric Light Co.	12
Sioux City Gas and Electric Light Co,	6
Simmons Hdw. Co., St. Louis, Mo.	30
Tama and Toledo Electric Railway and Light Co.	12
Tipton Lighting and Heating Co.	12
Traer Electric Co.	36
Union Electric Co., Dubuque, Ia	15
Waterloo and Cedar Falls Gas and E. Lt. Co.	24
What Cheer Electric Lt., & P. Co.	10

In the large majority of cases the number of sixteen candle power lamps of any one make sent in from one source does not exceed twelve.

The following table gives the sources from which more than one doxen sixteen candle power lamps of one make were received, together with the kind of lamp and the number sent in.

Cascade Milling Company	30	Edison
Citizens Light, Heat & Power Company	13	Majestic
City of Ames	24	Edison
City Water and Light Plant, Paris, Tenn.	32	Packard
Traer Electric Company	24	Miller Patent

It will be seen therefore, that the series of lamps tested represents fairly well the lamps commercially distributed throughout the state.

Tests Made. The tests made and recorded in this bulletin include tests for:—

Initial mean horizontal candle power.
Initial mean spherical candle power.
Initial watts per mean horizontal candle power.
Initial watts per mean spherical candle power.
Vacuum Tests.

Methods. The mean horizontal candle power measurements were made upon a standard Reichsanstalt photometer fitted with a Lummer-Brodhun screen. The secondary standards used were secured from the Electrical Testing Laboratories of New York city. These standards were used upon the photometer throughout the test, their permanence being determined by occasionally checking them against each other. The standard and test lamps were in all cases two meters apart. Stationary standards were used and the test lamp was rotated at approximately 180 revolutions per minute. In all measurements on this photometer readings were made with the screen direct and reversed and the average of these readings taken as the true setting of the screen.

The mean spherical candle power was measured by means of a Matthews' integrating photometer fitted with a Bunsen screen. The adjustment of this instrument was made in our laboratory and the results obtained by it were checked by a series of comparative tests by the Franklin Institute and Rousseau methods. Excepting those lamps which were designed to give high illumination in some one direction the results were found to check within from 2% to 5%. The Standards used were of the same nature as those employed in the mean horizontal measurements.

The power measurements were made by means of a Weston portable D. C. Voltmeter, checked by a Weston semi-portable laboratory standard, and a Weston portable D. C. Ammeter checked by means of the standard lamps which were certified as to current consumption.

The power used was obtained from a 60 cell, 120 ampere-hour storage battery for all tests excepting those of the 220 volt lamps. The voltage necessary for testing these lamps was secured by operating one of the 110 volt dynamos of the college lighting plant in series with the storage battery. The same electrical measuring instruments were used in all cases, a multiplier being placed in series with the voltmeter for all tests requiring over 150 volts.

For vacuum tests the induction coil method was employed.

Accuracy. It is believed that a fair degree of accuracy has been attained in these tests. The statement has been made on good authority that the maximum error in properly conducted commercial photometric measurements does not exceed 2% even when lamps are being photometered at the rate of 500 lamps per day. In the tests herein described the number of lamps per hour probably never exceeded fifteen. Great care was used in all adjustments and readings and to guard against the effects of eye fatigue, the operator upon the appearance of this difficulty, rested from his labors until such fatigue disappeared.

In view of these facts, it is reasonable to expect that the maximum error made in these tests is within the limits above indicated. In the results as set forth in the diagrams and tables two lots of lamps are included. In the tables they are distinguished as the old and new series. In the diagrams the lamps of the new series are represented by solid circles or dots, those of the old series by open circles.

The lamps of the old series were tested in part by four different individuals. The source of power was one of the dynamos of the college power plant and the electrical measuring instruments used were the best afforded by the Department of Physics and Electrical Engineering and were used almost exclusively for this purpose. No spherical measurements were made of this series.

The lamps of the new series were all tested for initial candle power and wattage by one individual. The source of power was the storage battery above described and the instruments after careful checking were used exclusively for this testing work. All spherical measurements were also made by the one observer.

RESULTS

The results of the tests are set forth in the following tables and target diagrams.

In Table No. 1 are given the average results of the initial candle power and wattage tests for each kind of lamp. The figures in the second column indicate the number of lamps upon which mean horizontal measurements were made. In every case in which figures are given for spherical measurements the number of lamps tested for mean horizontal and mean spherical candle power was the same within one or two lamps except in the case of the Columbia lamps. The number of lamps of this make tested for mean spherical candle power was 27. The general averages of all tests for initial candle power and wattage are given in Tables No. 2 to No. 5 according to the rated watts per candle power. The results shown in Tables No. 2 to No. 5 were obtained directly from Table No. 1. Table No. 3 shows the results of the "Old Series" tests and Table No 4 combines the results of Nos. 2 and 3

The selection of the various makes of lamps as given in Table No. 6 is specified by giving the number of lamps of each kind tested and the number falling within the target in the target diagram, that is, the number included within the prescribed limits as given above. The average selection results according to efficiency are set forth in Table No. 7. The results given in Tables No. 6 and 7 were obtained from the target diagrams which follow, by actual count.

In the vacuum tests, a lamp was classified as "medium" when the blue Geissler discharge was distinct and even quite marked and was yet not so full and complete as in those lamps classified as "low." In a sense this classification is misleading and it is possible a more consistent one would be obtained by combining the fourth and fifth columns of Table No. 8 under the heading "Poor Vacuum." This test included 2, 4, 8 and 32 candle power lamps in addition to the 16 candle power lamps discussed in this bulletin.

Target diagrams 1 to 14 inclusive give results of the initial mean horizontal candle power and wattage tests of the different makes of lamps for each efficiency, the figure placed upon the sloping efficiency line indicating in each case the rated watts per candle power of the lamps tested.

Diagrams 15 to 18 give the general results for all lamps of each efficiency tested. "Old Series" lamps are included in these diagrams and are indicated in all cases by open circles.

Diagrams 19 to 22 are drawn in the usual manner except that mean spherical instead of mean horizontal candle power is used. The lamps represented in these diagrams are in general the same as those of diagrams 7, 8, 9 and 13. The efficiency lines of these diagrams are, of course, based upon the horizontal rating.

These diagrams show in a general way the effect of changing the rating of a lamp from the mean horizontal to the mean spherical basis.

CONCLUSIONS

The general conclusions which may be drawn from the results of these tests are:

1. There is a wide variation in the incandescent lamps to be found upon the market,

    (a) In selection.
    (b) In actual candle power.
    (c) In actual wattage.
    (d) In vacuum,

and therefore in life, since, other things being equal, a low vacuum indicates a short life.

    (e) In "spherical reduction factor," i. e., the ratio of the mean spherical to the mean horizontal candle power.

2. About 62% of commercial sixteen candle power lamps are well selected.

3. About 76% of commercial incandescent lamps are properly exhausted

TABLE NO. VII

INITIAL CANDLE POWER AND WATTAGE TESTS

Kind of Lamp	Number Tested	Rated Watts per M. H. C. P.	Rated Voltage	Results of Tests Averages: Candle Power: Mean Horizontal	Results of Tests Averages: Candle Power: Mean Spherical	Watts per M. H. C. P.	Watts per M. S. C. P.
Am. Lamp Co.	4	3.5	104	14.66	12.25	3.61	4.21
Banner	21	“	104–115	18.18	15.31	3.12	3.67
Capital	4	“	100	11.57	9.86	4.53	5.43
Columbia	36	“	104–130	15.6	15.	3.62	3.77
Comet	6	“	110	18.36	14.83	3.11	3.82
Edison	59	“	100–112	15.9	13.37	3.53	4.15
Fostoria	14	“	104–110	14.44	12.56	3.78	4.31
Hall	3	“	108	16.6	....	3.45	....
Majestic	17	“	104–110	14.51	12.1	3.76	4.46
Miller Patent	32	“	104–120	15.45	12.89	3.87	4.62
Moline	12	“	104–115	16.05	13.69	3.51	4.07
Owensboro Lamp Co.	12	“	115–120	16.25	....	3.42	....
Packard	19	“	100–110	16.57	13.79	3.24	3.85
Peerless	13	“	110–115	16.10	13.55	3.65	4.29
Pioneer	6	“	108	16.47	....	3.44	....
Sawyer-Mann	43	“	100–115	16.	13.6	3.54	4.13
Shelby	5	‘	104–108	13.37	13.13	4.29	4.33
Star	15	“	104–115	16.97	14.28	3.54	4.19
Sunbeam N. T.	6	“	110	17.07	13.71	3.26	4.00
Banner	6	3.1	102	18.32	15.05	2.72	3.32
Columbia	13	“	108–110	16.03	15.1	3.2	3.39
Miller-Patent	24	“	120–125	17.2	14.45	2.95	3.49
Moline	4	“	105	17.54	15.2	2.96	3.36
Packard	30	“	120	16.11	....	3.09	....
Sawyer-Mann	11	“	110	15.46	13.1	3.32	3.87
Westinghouse	11	“	100	15.79	13.7	3.09	3.56
Sawyer-Mann	11	3.6	110	17.5	14.95	3.56	4.09

TABLE NO. 2.

INITIAL HORIZONTAL CANDLE POWER AND WATTAGE TESTS

NEW SERIES

RATED EFFICIENCY	NO. LAMPS TESTED	RESULTS OF TESTS AVERAGES	
		MEAN HORIZONTAL CANDLE POWER	WATTS PER M. H. C. P.
3.1	99	16.44	3.07
3.5	327	15.88	3.54
3.6	11	17.50	3.55
3.8	23	15.00	4.15

TABLE NO. 3.

INITIAL HORIZONTAL CANDLE POWER AND WATTAGE TESTS

OLD SERIES

RATED EFFICIENCY	NO. LAMPS TESTED	RESULTS OF TESTS AVERAGES	
		MEAN HORIZONTAL CANDLE POWER	WATTS PER M. H. C. P.
3.1	20	15 31	3.32
3.5	51	15.29	3.71
4.0	15	15.12	4.25

TABLE NO. 4.

INITIAL HORIZONTAL CANDLE POWER AND WATTAGE TESTS

ALL LAMPS TESTED

RATED EFFICIENCY	NO. LAMPS TESTED	RESULTS OF TESTS AVERAGES	
		MEAN HORIZONTAL CANDLE POWER	WATTS PER M. H. C. P.
3.1	119	16.26	3.11
3.5	378	15.8	3.56
3.6	11	17.50	3.55
3.8	23	15.00	4.15
4.0	15	15.12	4.26

TABLE NO. 5.

INITIAL SPHERICAL CANDLE POWER AND WATTAGE TESTS

NEW SERIES

RATED EFFICIENCY	NO. LAMPS TESTED	RESULTS OF TESTS AVERAGES	
		MEAN SPHERICAL CANDLE POWER	WATTS PER M. S. C. P.
3.1	71	14.13	3.56
3.5	296	13.5	4.11

TABLE NO. 6

SELECTION

Kind of Lamp	Efficiency	No. Tested	No. Falli'g Within Target	Kind of Lamp	Efficiency	No. Tested	No. Falli'g Within Target
Banner	3.5	21	7	Owensboro Lamp Co.	3.5	12	10
Columbia	3.1	13	9	Packard	3.1	30	22
Columbia	3.5	36	19	Packard	3.5	19	9
Edison	3.5	59	54	Peerless	3.5	13	2
Fostoria	3.5	14	6	Sawyer-Mann	3.1	16	8
Majestic	3.5	17	11	Sawyer-Mann	3.5	45	29
Miller-Patent	3.1	24	13	Sawyer-Mann	3.6	11	2
Miller-Patent	3.5	30	10	Star	3.5	15	6
Moline	3.5	12	9	Westinghouse	3.1	11	11

TABLE NO. 7.

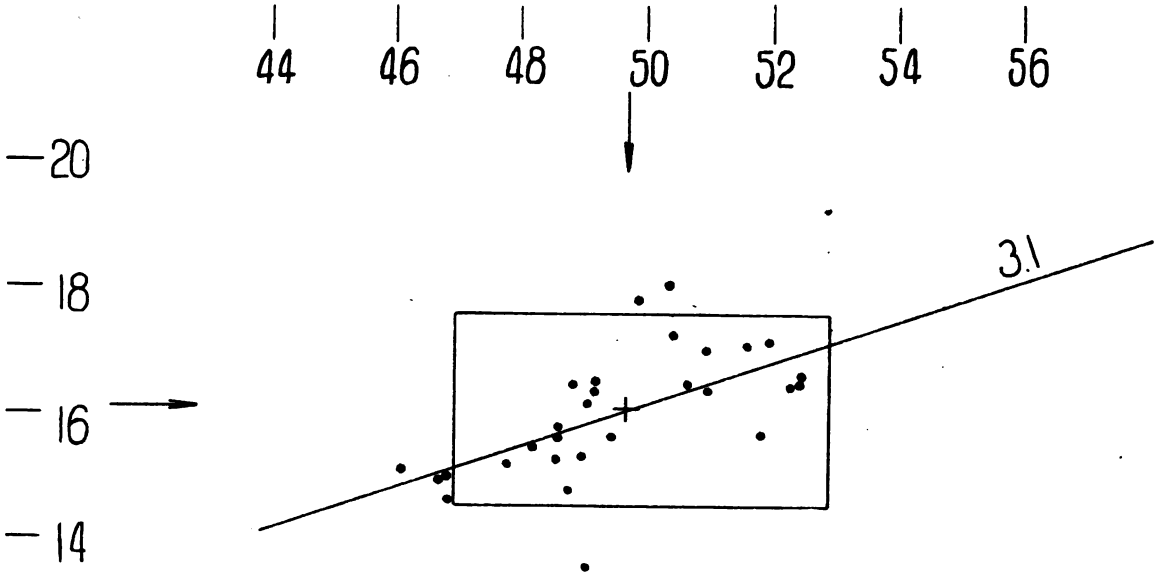
SELECTION

Kind of Lamp	No. Tested	No. Falling Within Target	Selection per cent
All 3.1 Watt Lamps	138	90	65.1
All 3.1 Watt Lamps New Series	118	79	67.0
All 3.5 Watt Lamps	385	238	62.0
All 3.5 Watt Lamps New Series	335	212	63.3
All 3.6 Watt Lamps	11	2	18.2
All 3.8 Watt Lamps	23	13	56.5
All 4.0 Watt Lamps	15	9	60.0

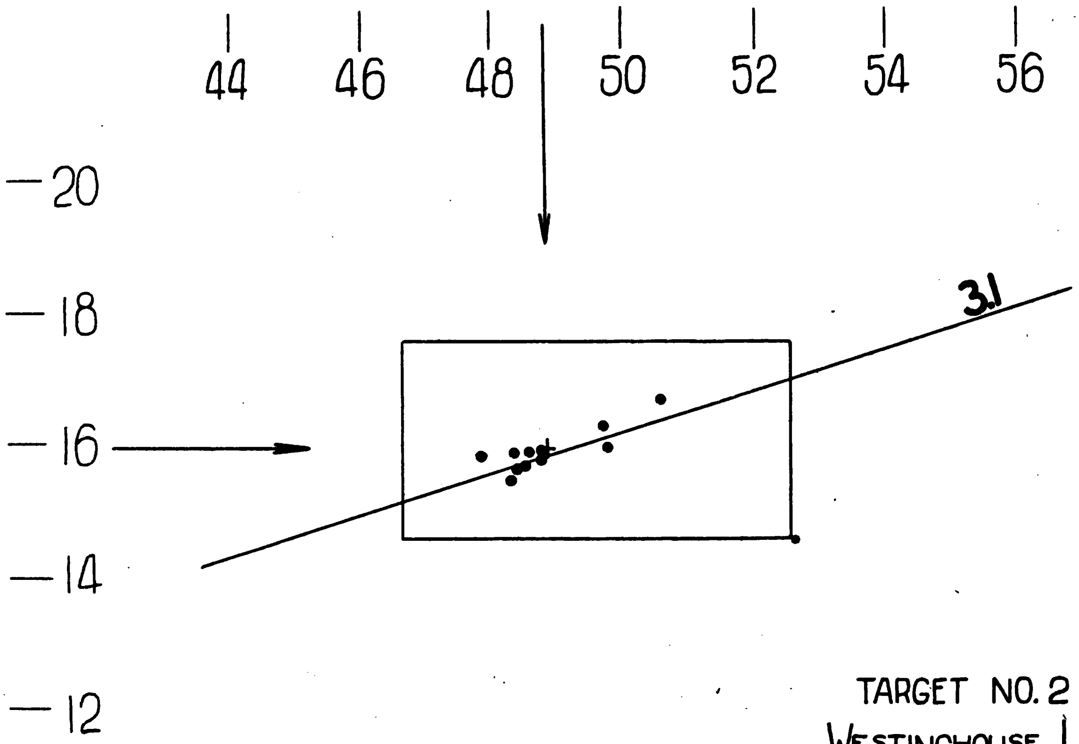
TABLE NO. 8.

VACUUM TESTS

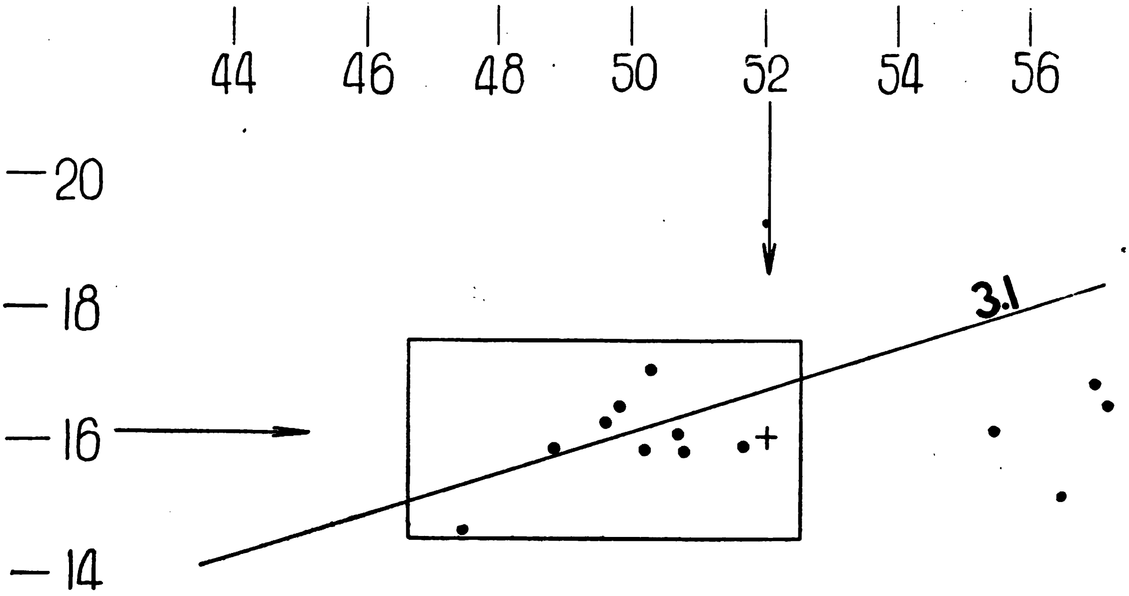
Kind of Lamp	Number Tested	VACUUM		
		Good	Medium	Low
Am. Lamp Co.	6	4	1	1
Banner	49	43	4	2
B. G. & E. Co.	2	2		
Bright	2		1	1
Capital	8	6	1	1
Columbia	65	37	12	16
Comet	6	6		
Commercial	1		1	
Edison	132	92	23	17
Equality	1	1		
Fostoria	27	19	5	3
Gilmore	....	....	....	....
G. E.	....	....	....	....
Hall	1	1		
Imperial	5	4	1	
Manhattan	1			1
Majestic	26	26		
Miller Patent	77	67	2	8
Moline	24	13	2	9
N. E. Co.	12	12		
Owensboro Lamp Co.	12	12		
Packard	27	22	1	4
Peerless (Ed)	21	17	1	3
Radium	3	2	1	
Sawyer-Mann	90	65	17	8
Shelby	9	8		1
Star	19	12	2	5
Sterling	1	1		
Sunbeam (N. T.)	6	6		
Sunshine	2	2		
U. S.	4	4		
Westinghouse	12	12		
Total	651	496	75	80
Percentages	100	76.1	11.5	12.3



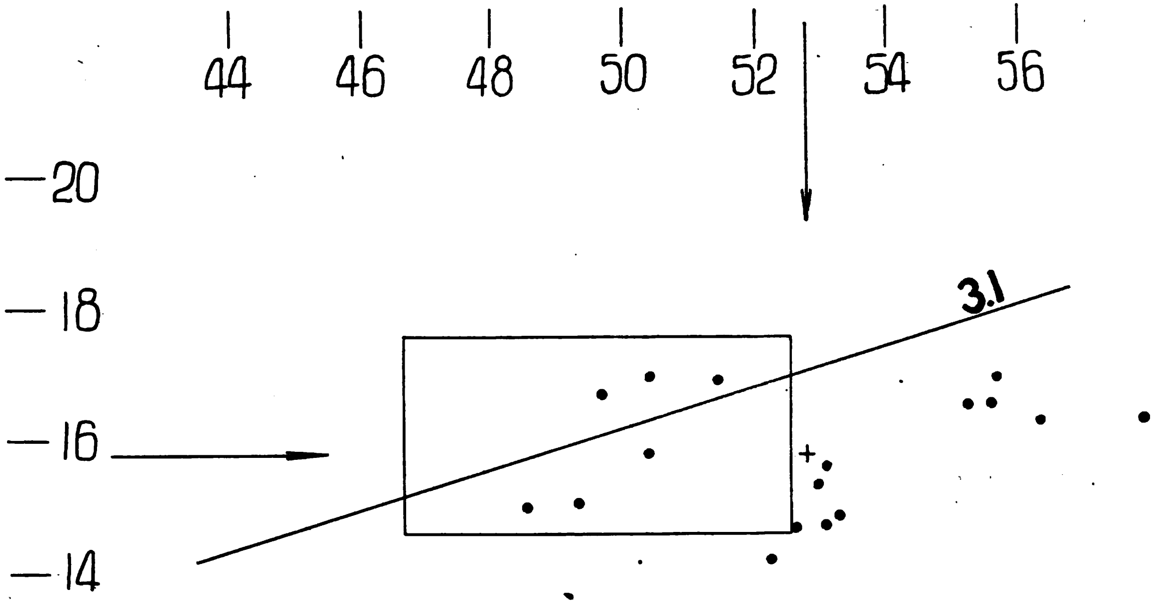
TARGET NO. 1
PACKARD LAMPS



TARGET NO. 2
WESTINGHOUSE LAMPS



TARGET NO.3
COLUMBIA LAMPS



TARGET NO.4
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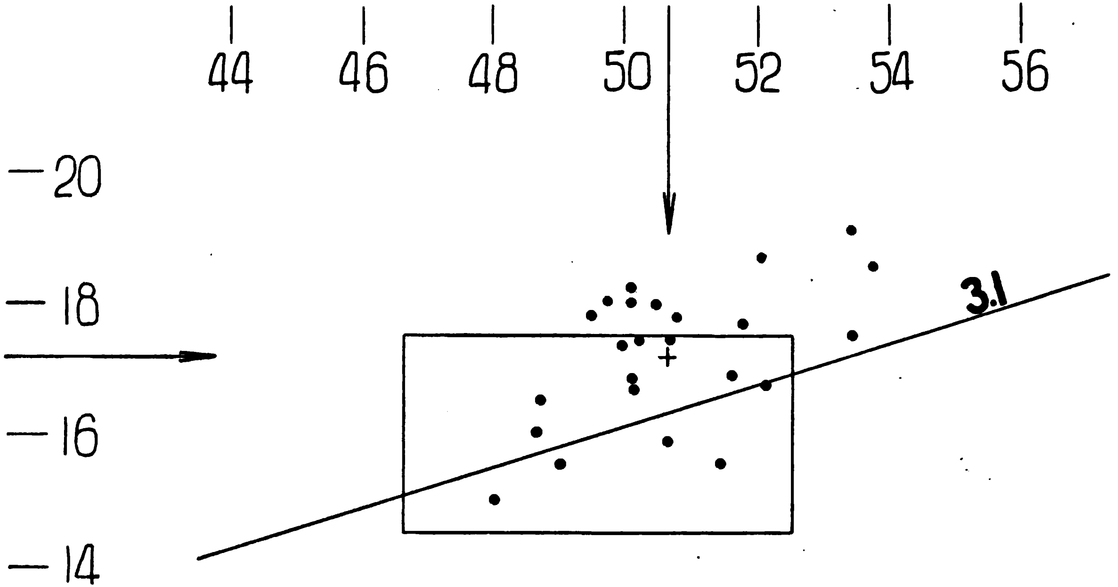
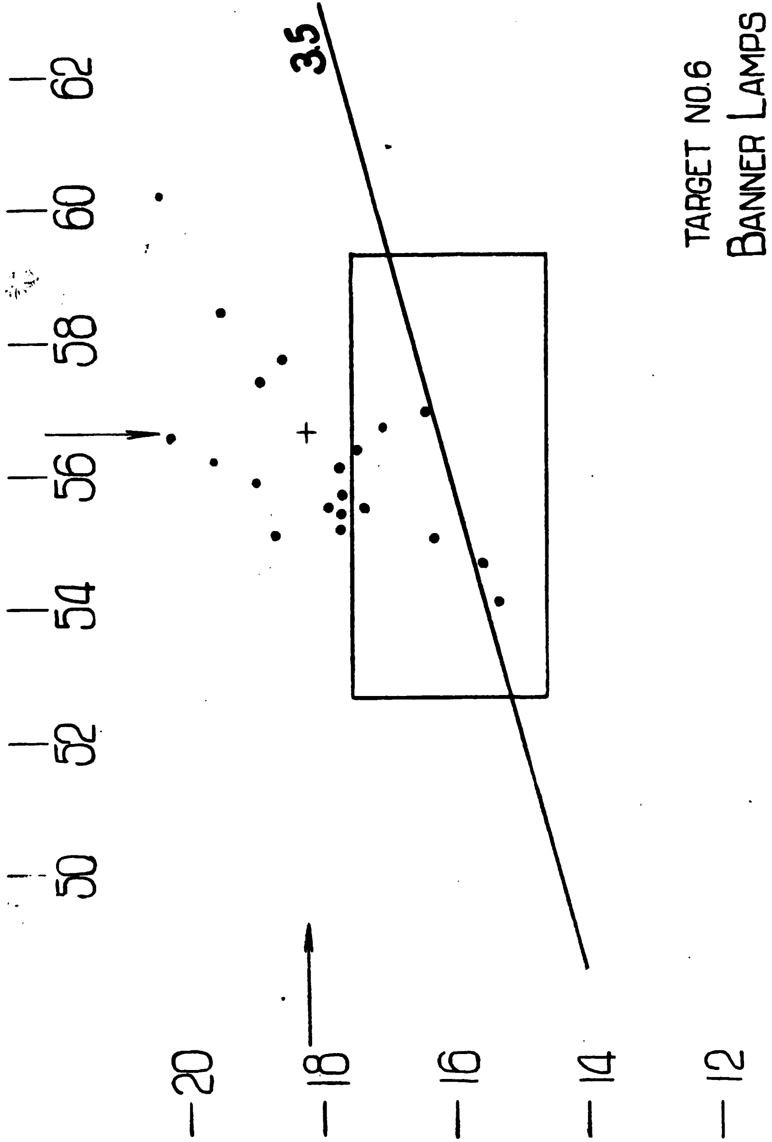
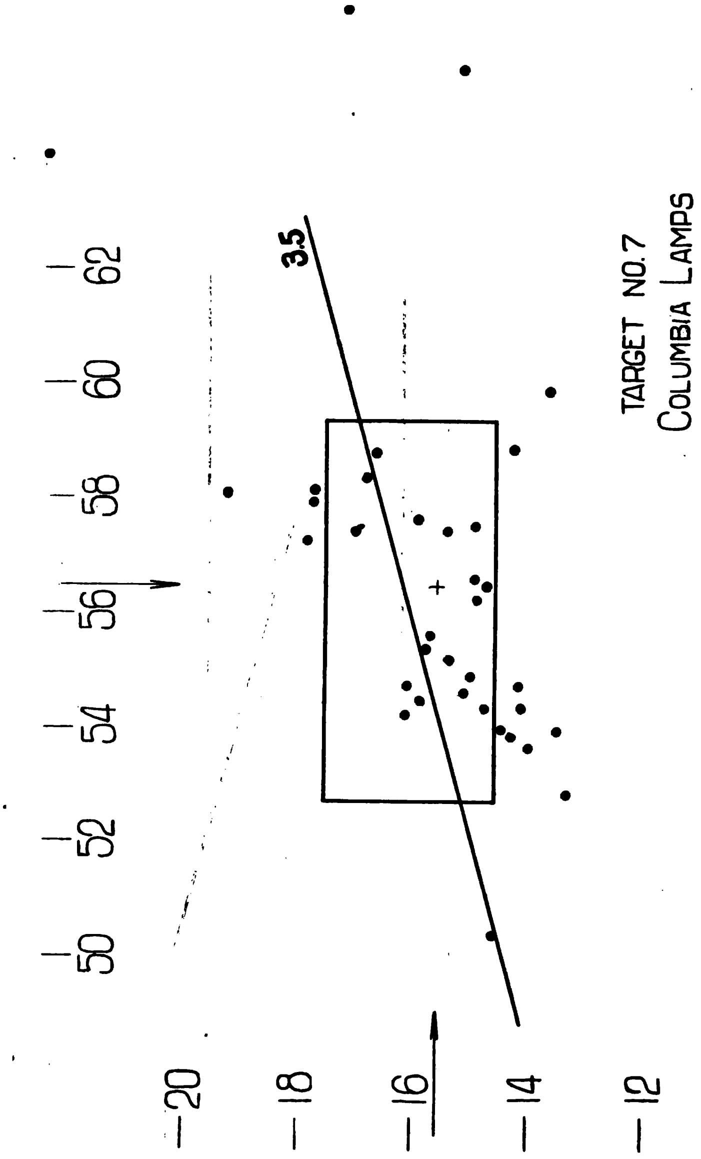


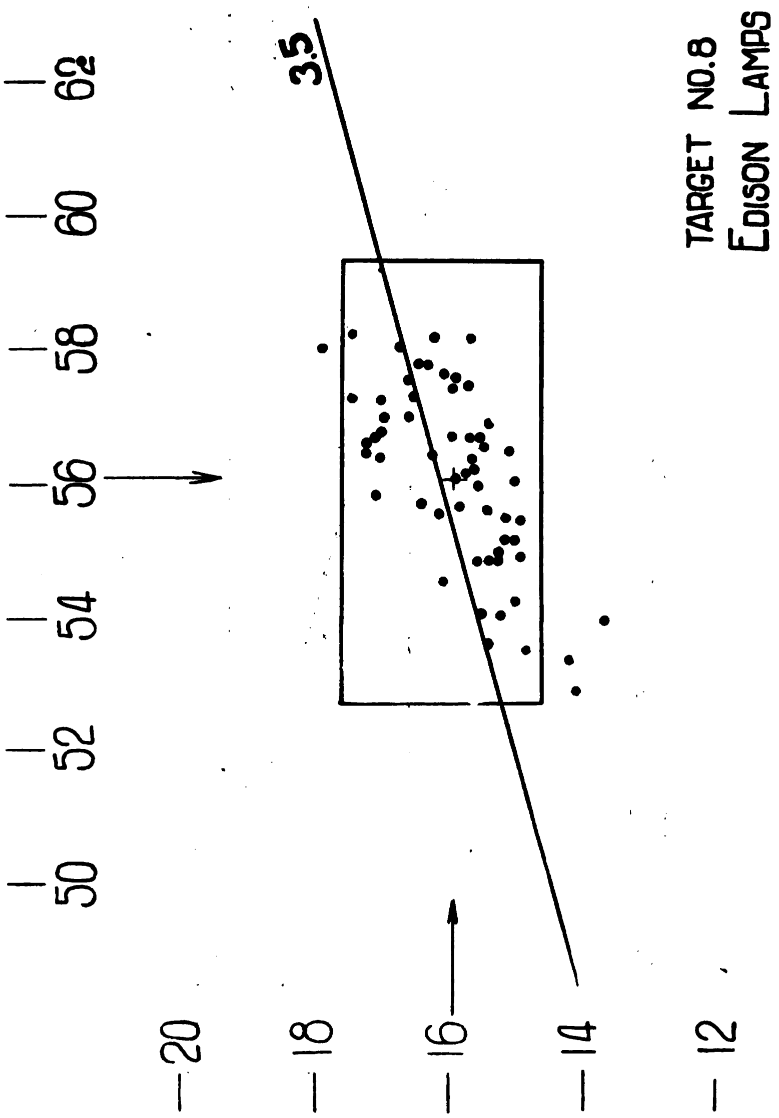
TARGET NO. 5
MILLER-PATENT LAMPS

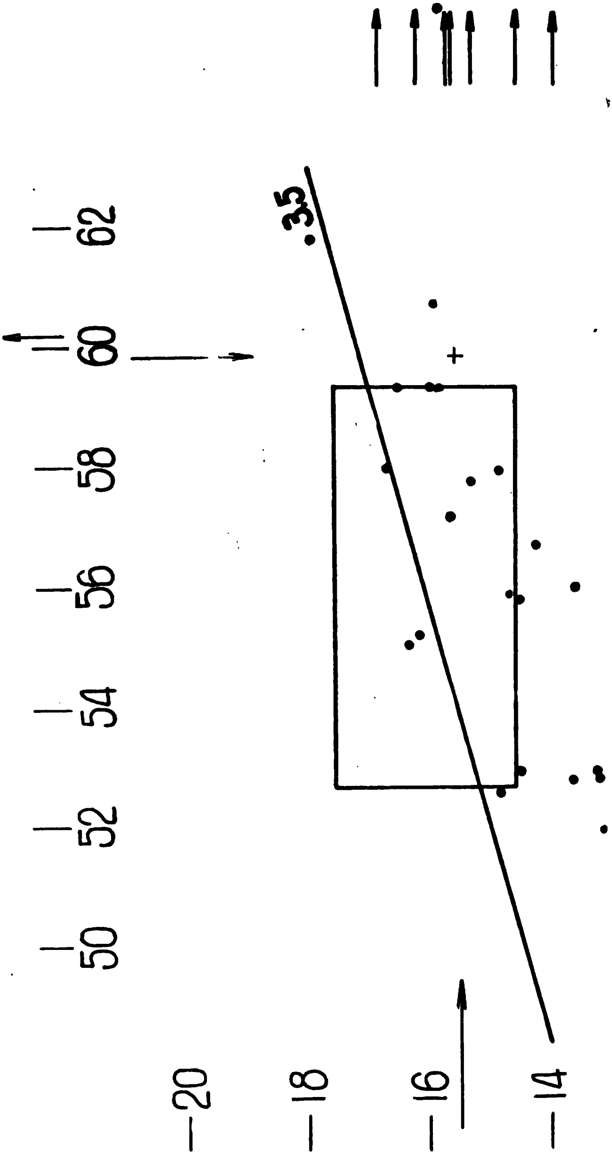


TARGET NO.6
BANNER LAMPS

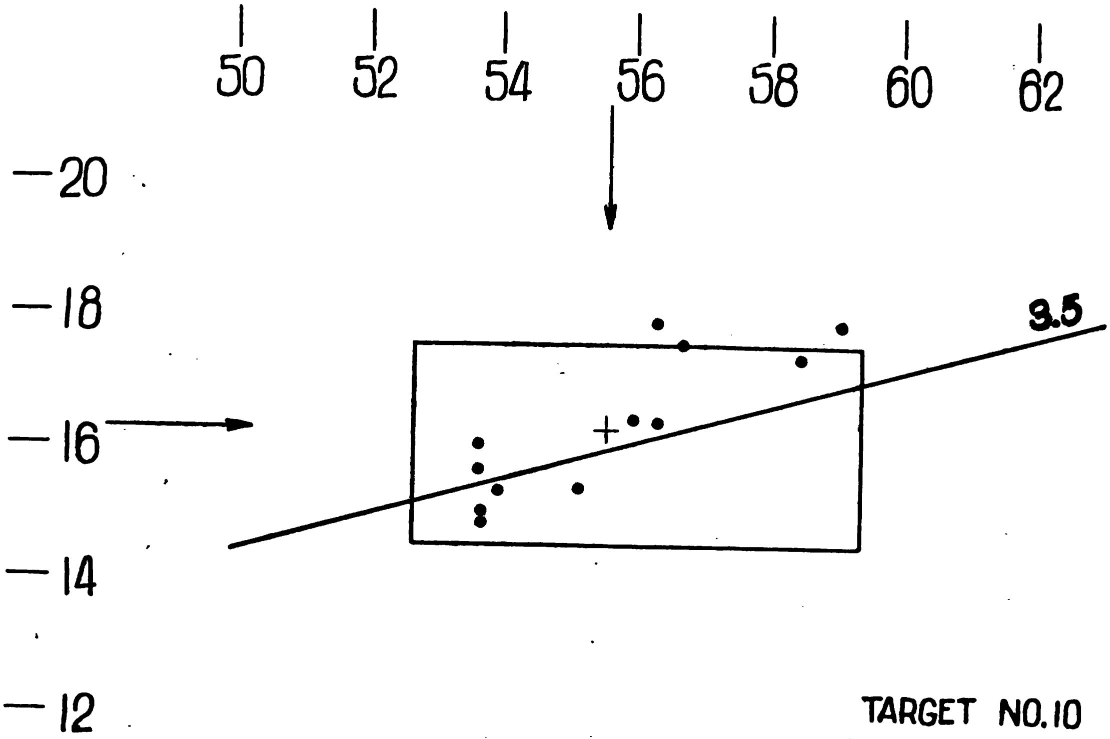


TARGET NO.7
COLUMBIA LAMPS

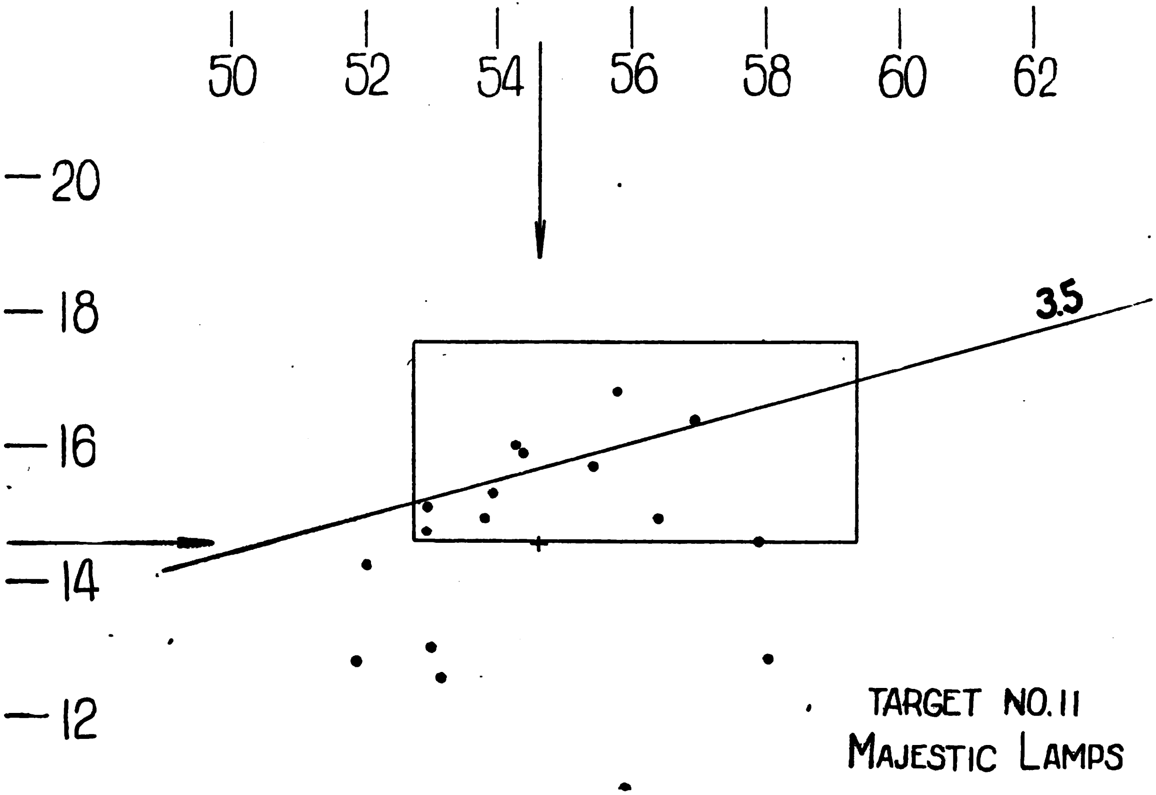

TARGET NO.8
EDISON LAMPS
3.5
50
52
54
56
58
60
62
-20
-18
-16
-14
-12



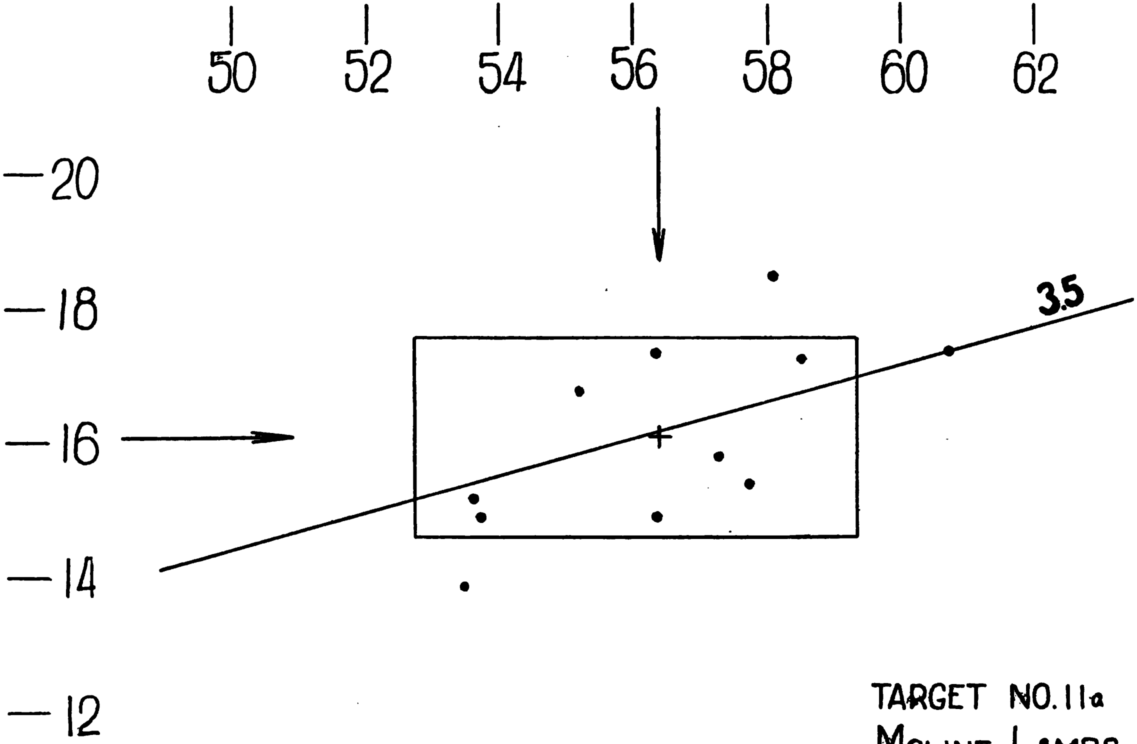
TARGET NO.9
MILLER PATENT LAMPS



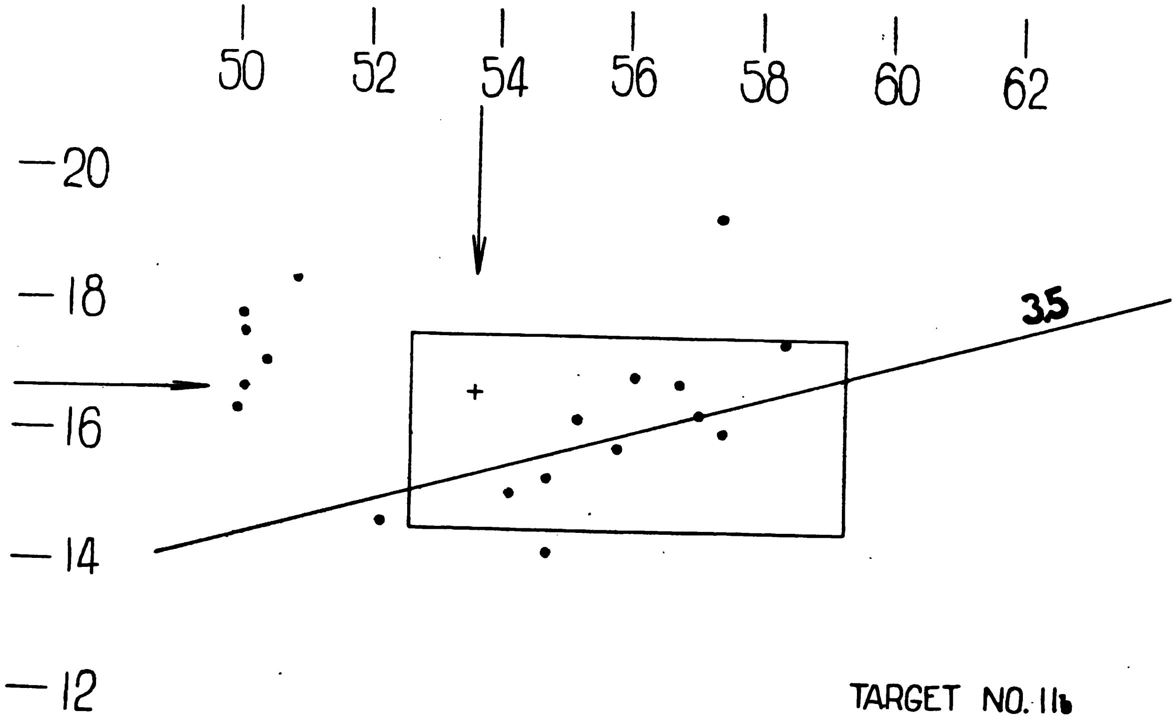
TARGET NO.10
OWENSBORO LAMPS



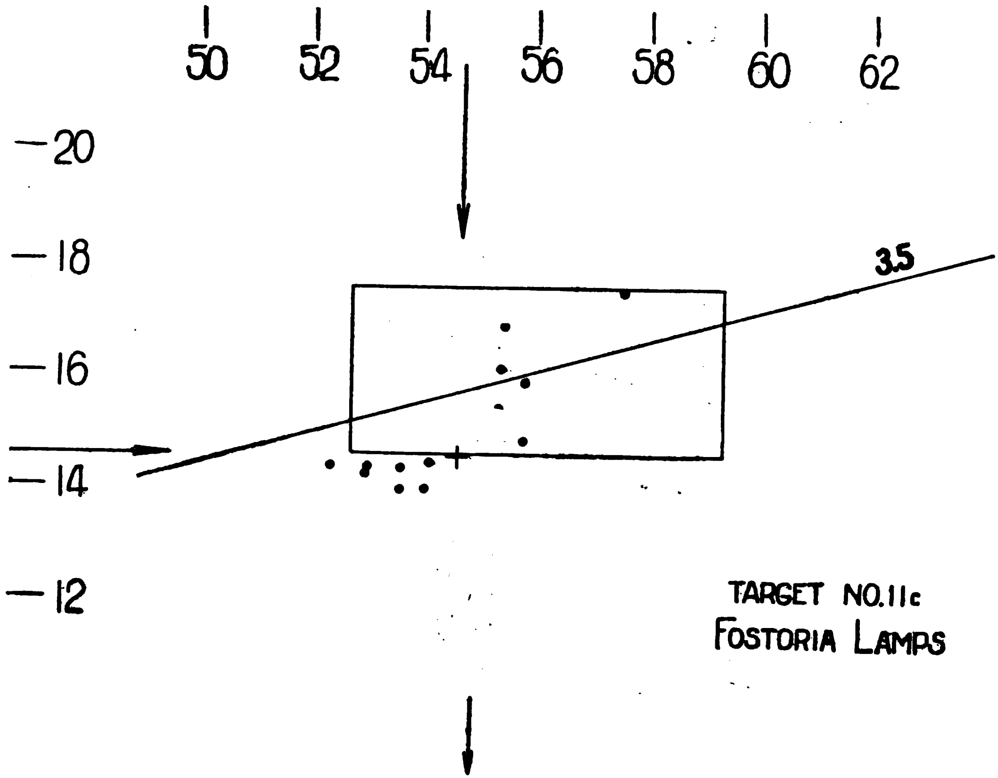
TARGET NO. 11
MAJESTIC LAMPS



TARGET NO. 11a
MOLINE LAMPS



TARGET NO. 11b
PACKARD LAMPS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52
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62
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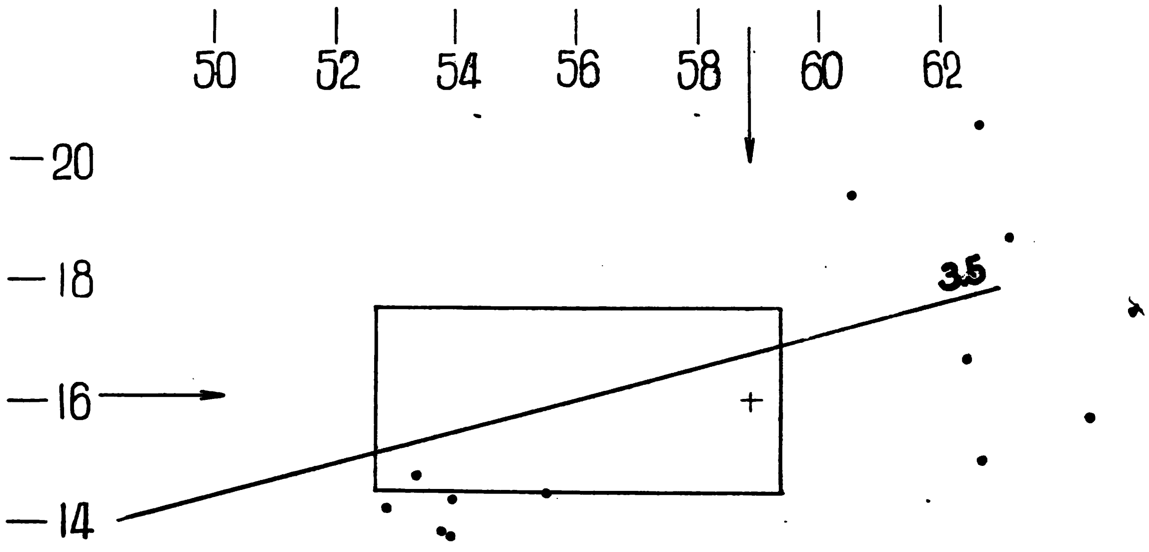
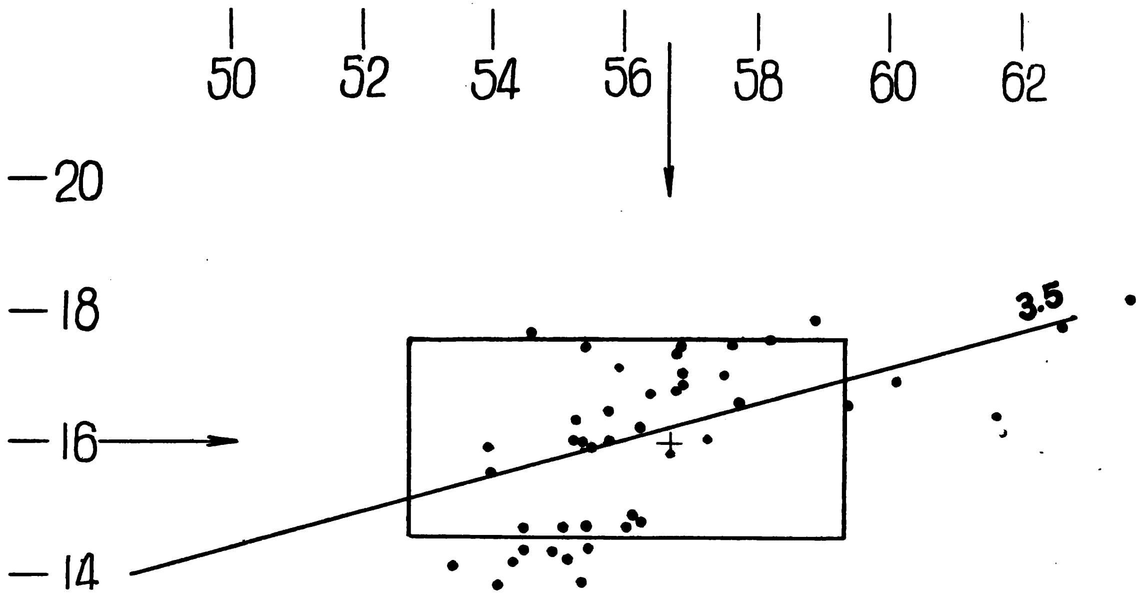
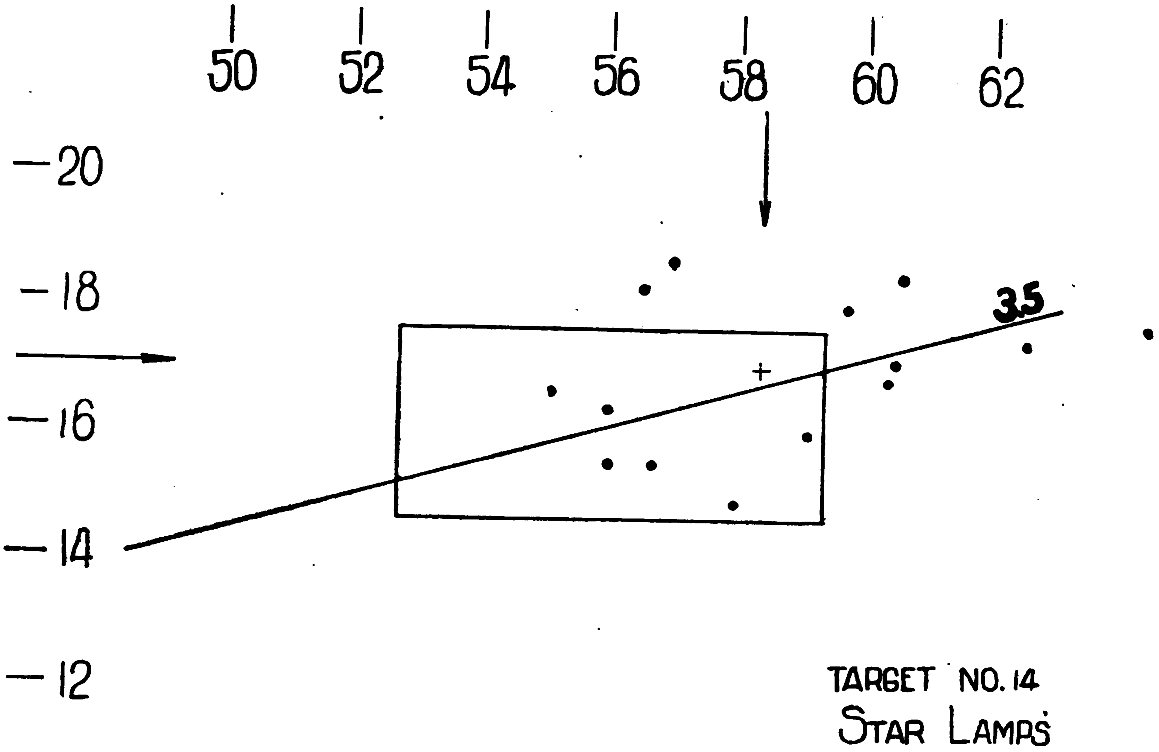
16
14
12
3.5
TARGET NO.11c
FOSTORIA LAMPS



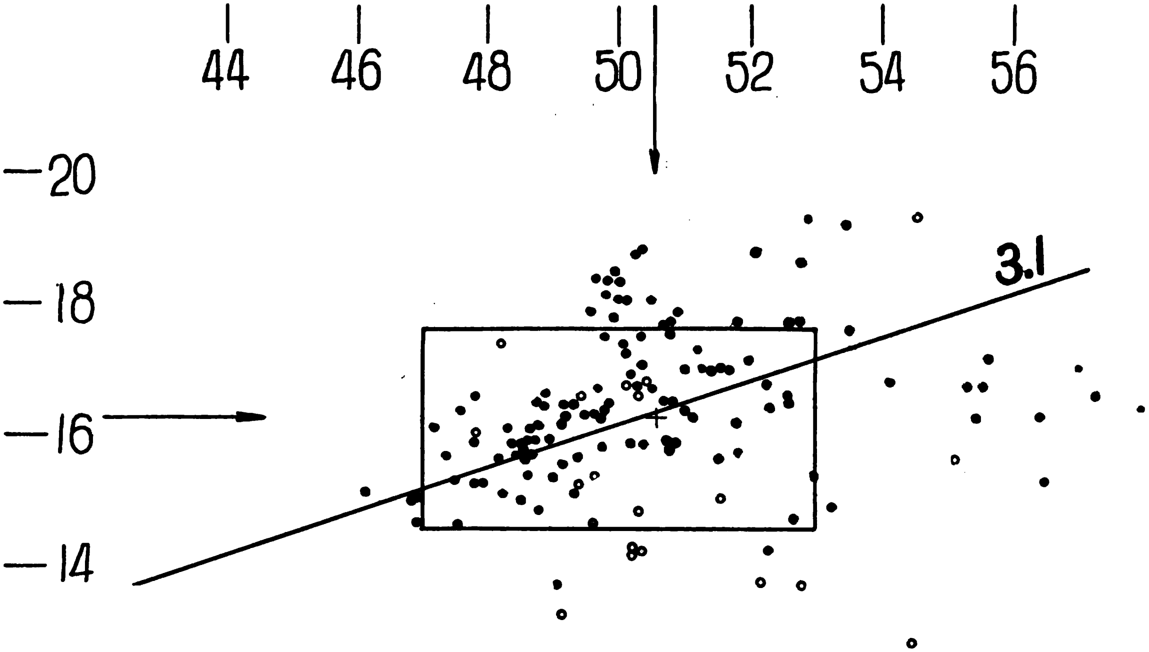
TARGET NO.12
PEERLESS LAMPS



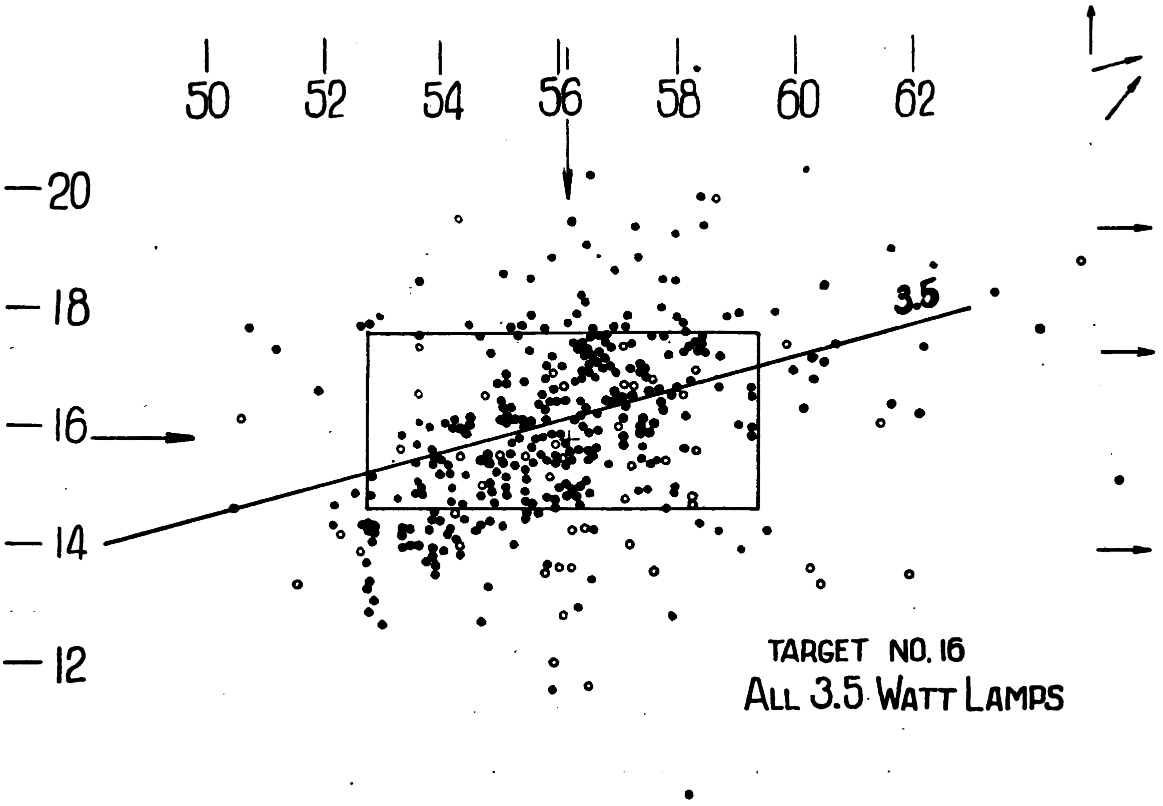
TARGET NO. 13
SAWYER-MANN LAMPS



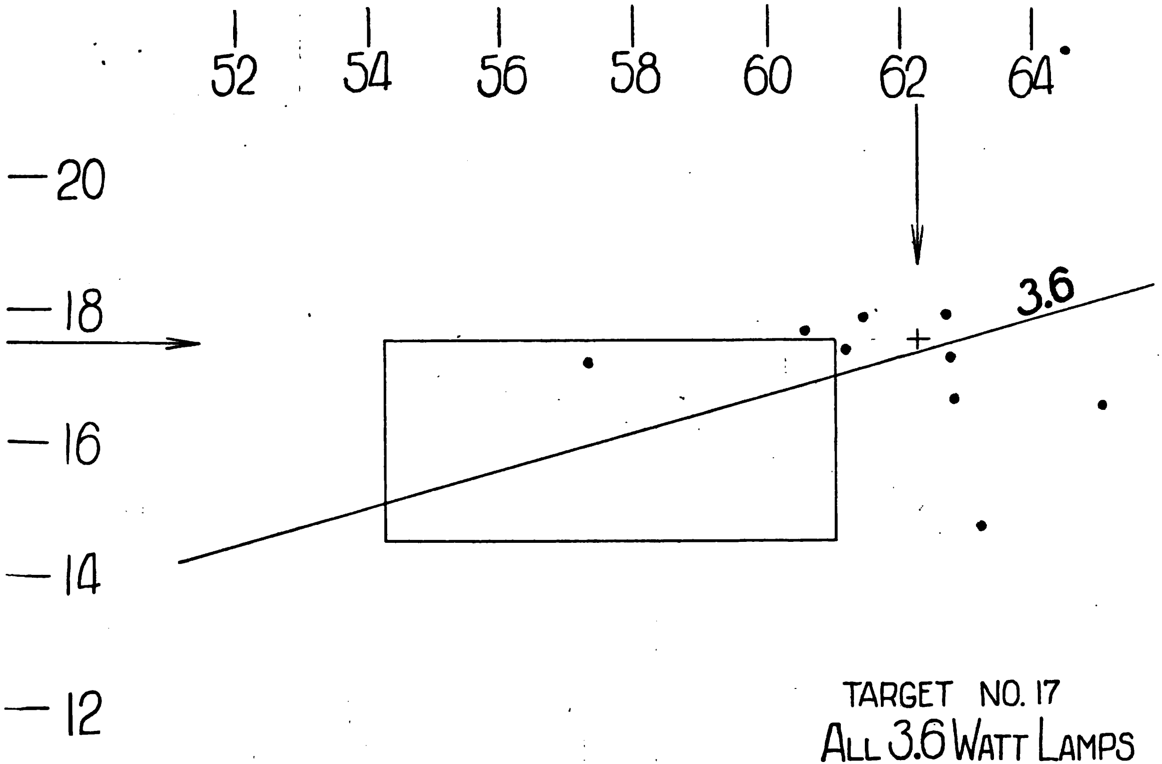
TARGET NO. 14
STAR LAMPS



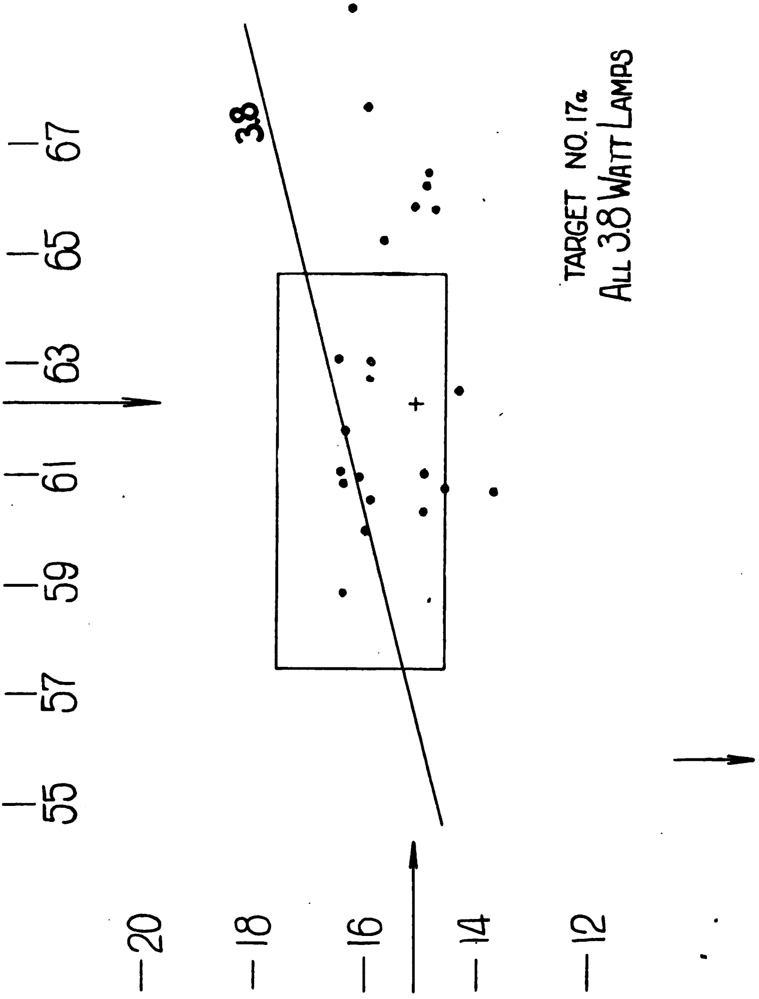
TARGET NO. 15
ALL 3.1 WATT LAMPS

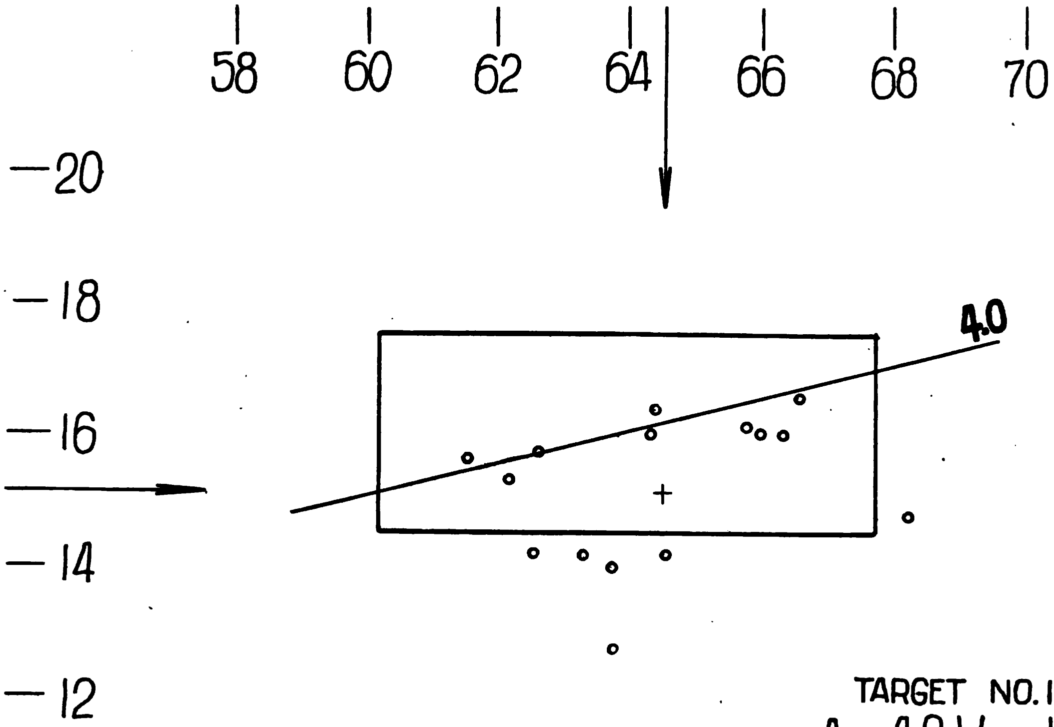


TARGET NO. 16
ALL 3.5 WATT LAMPS

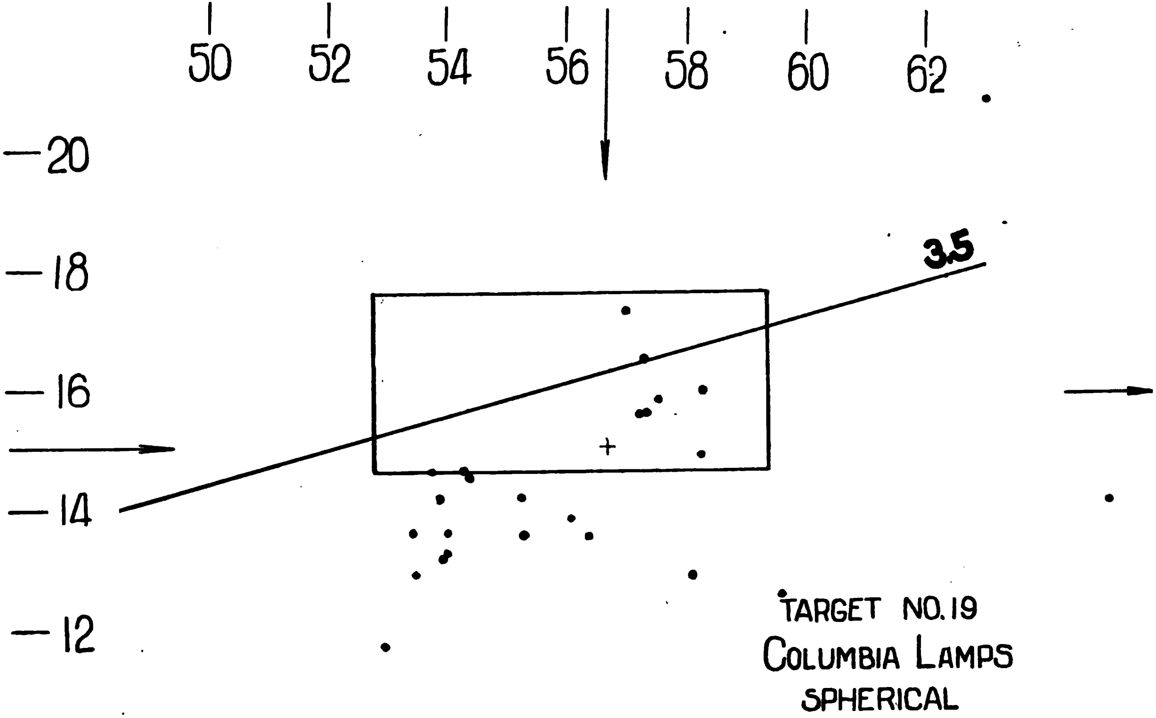


TARGET NO. 17
ALL 3.6 WATT LAMPS

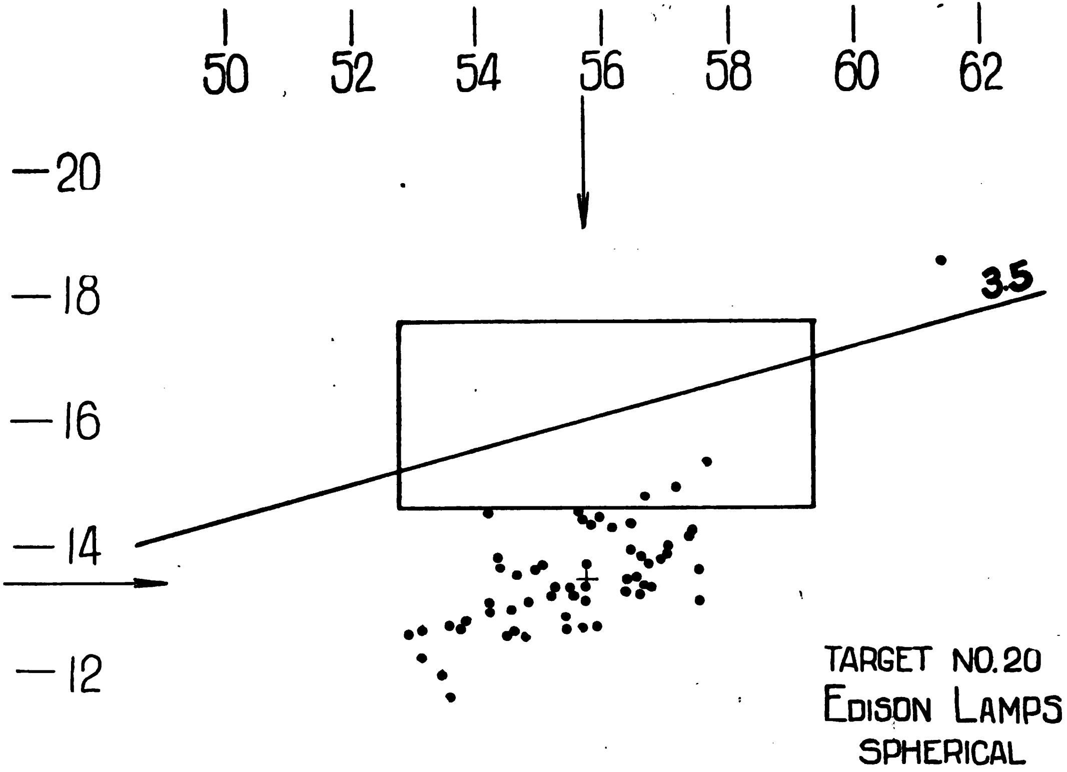

55
57
59
61
63
65
67
−20
−18
−16
−14
−12
3.8
TARGET NO. 17a
ALL 3.8 WATT LAMPS



TARGET NO. 18
ALL 4.0 WATT LAMPS



TARGET NO. 19
COLUMBIA LAMPS
SPHERICAL


50
52
54
56
58
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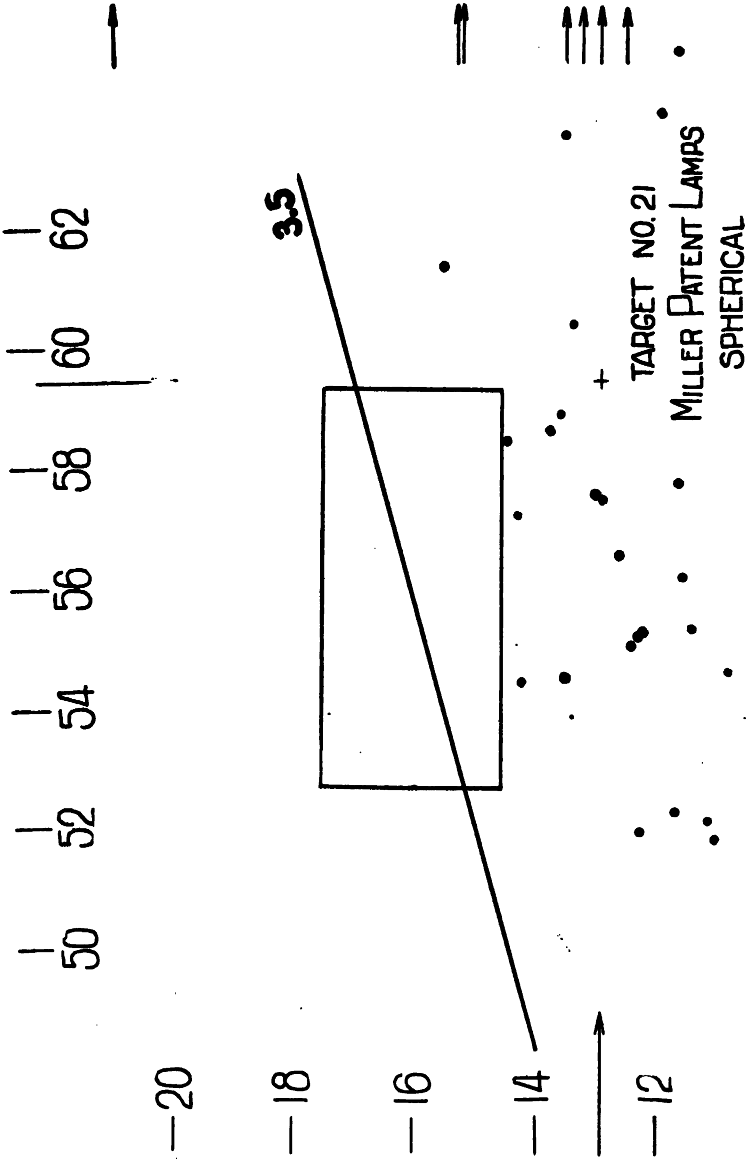
62
20
18
16
14
12
3.5
TARGET NO. 20
EDISON LAMPS
SPHERICAL


50
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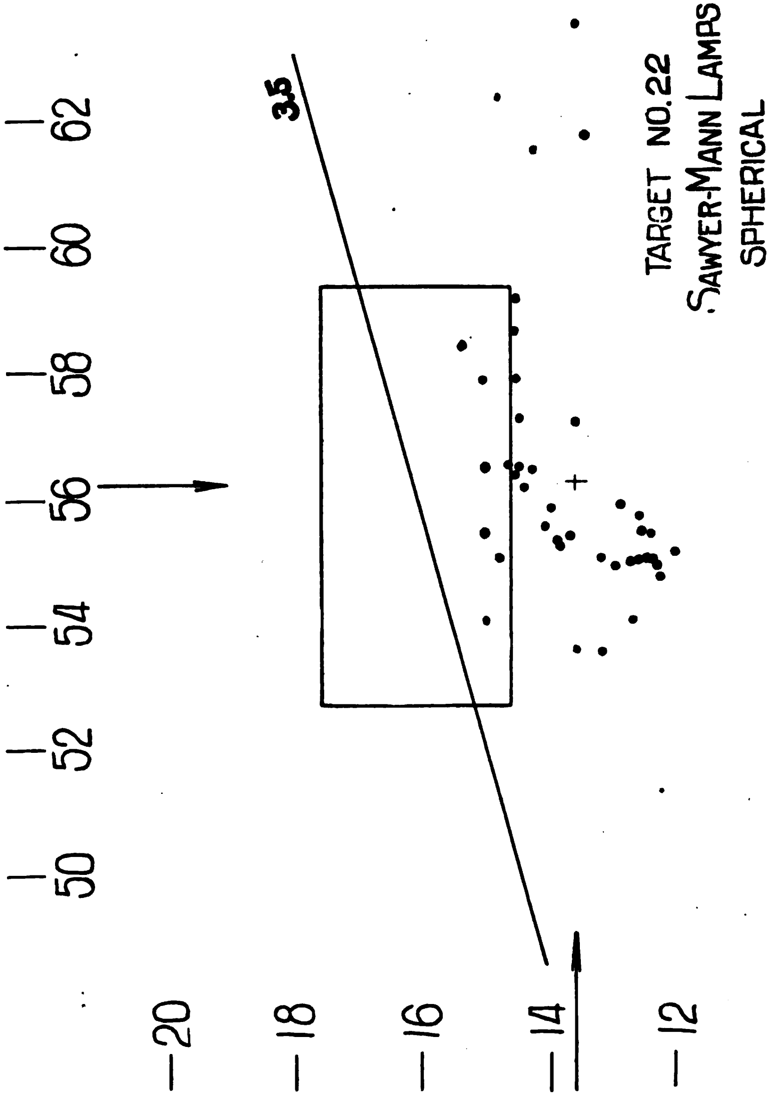
58
60
62
20
18
16
14
12
3.5
TARGET NO. 21
MILLER PATENT LAMPS
SPHERICAL


TARGET NO.22
SAWYER-MANN LAMPS
SPHERICAL
3.5
50
52
54
56
58
60
62
-20
-18
-16
-14
-12

Engineering Experiment Station

IOWA STATE COLLEGE

Ames, Iowa

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ASSOCIATE PROFESSOR W. H. MEEKER	Mechanical Engineering

BULLETIN

VOL III No. 5

Mechanical Engineering Section

Efficiency of Steam Pipe Coverings

APRIL 1906

Introduction

The tests which are reported in this Bulletin were originally conducted as college work rather that the work of the Engineering Experiment Station. The results being so consistent as to indicate accuracy in the carrying on of the work, and the subject being one of utmost importance to all steam users, it was considered desirable to publish the results in the form of a bulletin.

The work was carried on by Messrs. A. Cammack and F. E. Woodman, graduates in Mechanical Engineering in 1905. This bulletin is almost an exact reproduction of their thesis, except that the curves were platted by the undersigned from their tables. The economic value of the covering is shown in Table 6. The cost of covering as used in computing these valued is given in the right hand column of Table 5, and is in accordance with the list price of the manufacturers. They recommend that the cost of freight and applying the covering be taken as equal to the discount from the list price in making estimates, thus the use of these figures.

Iowa State College,
April, 1906.

W. M. Wilson.

Economy Test of Steam Pipe Covering for Different Sizes of Pipe

The object of this test was to investigate the economy of steam pipe coverings, both single and double, for pipes varying in size from 1 inch to 7 inches, and under pressures from 25 to 100 pounds by gauge.

The condensation method was used. In this method the pounds of steam condensed, the steam pressure, and the calorimeter and condensed steam temperatures being observed, the B. T. U.'s lost per pound by radiation is readily calculated from the steam tables.

APPARATUS

The pipe upon which the tests were to be made was connected up as shown in the accompanying sketch, Fig 1, all connections being made as short as possible and covered with a one inch cover of the same material as used in the rest of the test. This covering was left on the connections and caps at the end of the pipe throughout the test of each size of pipe, and corrections were made for the radiation from these connections. The connections through which the steam enters the pipe were all one inch, while those at the other end were half inch. The lengths of the different sizes of pipe ranged from sixteen to nineteen feet. These pipes were placed with a slope of about ten inches in eighteen feet. The condensation was collected in the system of small pipes at the lower end of the test pipe, the water surface being kept at a constant level by means of a globe valve and water glass. The temperature of the water drawn off was obtained by use of a thermometer inserted into a thermometer well which extended down inside the pipe so as to be surrounded at all times by the water within the pipes.

The temperature of the room was determined by six thermometers placed along and about three feet from the pipe, three on a side.

The condensation as it was let out through the globe valve,

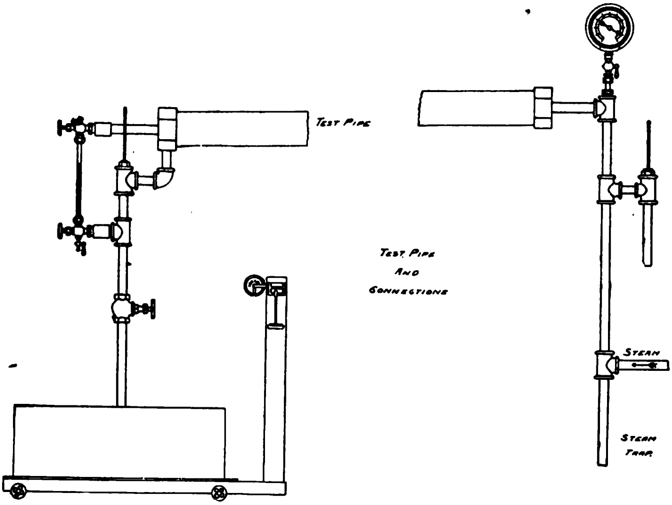

FIGURE 1

was collected beneath the surface of cold water. A barrel partly filled with cold water, and resting upon a platform scale, was used for a collecting tank. The barrel, being entirely closed except for the small opening where the drain pipe entered, the chance for evaporation was reduced to a minimum.

THE TEST

Steam was admitted to the pipe by opening the globe valve in the small steam pipe, the desired pressure being obtained by throttling at the valve. Steam was admitted to the pipe long enough before the test so that the pipe or pipe and coverings, became thoroughly warmed, and the flow of condensation regular. The water of condensation was brought to the desired height in the water glass at the beginning of the run, care being taken to have it at the same height at the end of the run. Readings of the temperatures of the room, calorimeter, and condensed steam were taken every five minutes, and the condensation was weighed every fifteen or twenty minutes.

The lengths of the runs were made such that the amount of condensation was nearly the same in each case, and varied from one and a half to four hours. To eliminate any error that would result, the room in which the test was made was kept entirely free from outside draughts, and the water was weighed carefully on a scale reading to one one-hundredth of a pound.

Three sets of runs were made on each size of pipe, the first set being made on the bare pipe at pressures of 25, 50, 75 and 100 pounds, by gauge; the second set with a single cover in the pipe and the same range of pressures; and the third set with a double cover on the pipe and the same range of pressures. A log was made for each run, a sample of which is given below.

The commercial form of Keasbey & Mattison 85% Magnesia Sectional Covering, of standard thickness, was used throughout the tests, so that the results obtained on the different sizes of pipe are comparable

DISCUSSION OF RESULTS

A comparison of the results as given in Table No. 5 shows a gradual increase in the efficiency with the increase in the size of

No. 46

ECONOMY TEST OF STEAM PIPE COVERING

DEPARTMENT OF MECHANICAL ENGINEERING IOWA STATE COLLEGE

Size of Pipe, 7 inch. Form of Covering, Double. Pressure, 50 lbs.

Date, May 16, 1905. A. Cammack—F. E. Woodman.

	Pres-sure	Wgt.	TEMPERATURES									
Time	By Gaug'	Cond. Steam	Room						Cond. Steam	Cal.	Quality of Steam	Temperature of Steam
1.10	50		86.	85.	84.	85.	84.	85.	259	258.		
1.15	50		86.5	85.	84.5	85.	84.5	85.	262	258		
1.20	54		86.5	86.	85.	85.	85.	86.	271	260		
1.25	50	.40	87.	86.	85.	85.	85.	86.	266	260		
1.30	50		87.	86.	85.	86.	85.	86.	253	261		
1.35	53		87.	86.	85.5	86.	85.	86.	280	263		
1.40	52	.95	87.	86.	85.	86.	85.	86.	276	264		
1.45	52		87.	86	85.	85.	85.	86.	277	265		
1.50	43		86.5	85.5	85.	85.5	85.	86.	280	268		
1.55	58	1.46	87.	85.5	85.	85.	85.	86.	271	265		
2.00	50		87.	85.	85.	85.	84.5	85.	277	266		
2.05	50		86.	84.5	84.	85.	84.	85.	273	264		
2.10	40	1.95	86	84.	84.	85.	84.	84.	271	266		
2.15	48		86.	84.5	84.	85.	84.	84.5	273	263		
2.20	47		86.	84.5	84.	85.	84.	85.	282	262		
2.25	59	2 55	85.5	24 5	84.	85.	84.	85.	280	262		
2.30	40		86.	85.	84.	85.	84.	85.	281	263		
2.35	43		86.	85.	84.	85.	84.	85.	282	263		
2.40	48	3.18	86.	85.	84.	85.	84.	84.5	276	263		
2.45	58		85.5	84.	83.	85.	83.5	84.	277	261		
2.50	50		85.	84.	83.	84.5	83.5	84.	279	262		
2.55	50	3.76	85.	84.	83.	85.	83.	83.5	278	261		
3 00	48		84.	84.	83.	84.5	83.	83.5	276	259		
3.05	51		85.	84.	83.	84 5	83.	84.	277	260		
3.10	50	4.35	84.5	84.	83.	84.	83.	83.5	276	260		
	50	4.35	86.04	84.92	84.16	85.0	84–16	84 94	273.68	262.28	99.9	297.5

84.82

Length of Run=2 hrs. Difference in Temperature=212.63°

B. T. U. loss per lb.=927.4 Total=4034 B. T. U.

B. T. U. loss per hr. per degree, diff. in Temp. per sq. ft. of surface=.2585

the pipe, but it does not show, as was expected, any increase in the efficiency of the double over the single cover with the increase in size of pipe. It was thought that a point would be reached with the smaller pipes where the increased surface for radiation due to the coverings would offset the gain due to the poor conductivity of the coverings. The approximate ratio of the radiating surface of the bare pipe to that with a single cover for the four sizes of pipe is as follows:

For the 1 inch pipe 1:3
For the 3 inch pipe 3:5
For the 5 inch pipe 5:7
For the 7 inch pipe 7:9

FOR THE DOUBLE COVER

For the 1 inch pipe 1:5
For the 3 inch pipe 3:7
For the 5 inch pipe 5:9
For the 7 inch pipe 7:11

The results show that while the ratio of the radiating surface of the bare pipe and single cover has changed from 1:3 to 7:9, the efficiency of the covering has increased from 75.100 to 86.100, a comparatively slight increase as compared with the increased ratio of the bare pipe surface to that with a single cover. With the double covering the results show that the increased efficiency of the double cover over that of the single is almost a constant, in spite of the fact that the radiating surface of the small pipe has been increased a great deal more proportionately than for the larger pipes.

The efficiencies found for this grade of covering are almost identical, for the same size of pipe and the same conditions, with those found by previous tests made by either the condensation or electrical method.

Mr. H. A. Stott in a paper presented at the Twenty-third Convention of the Association of Edison Illuminating Companies, gives the report of tests of pipe coverings which he conducted. Among the coverings tested were two 85% Magnesia Coverings. The tests were conducted by the electrical method. The covering was 1 ⅛ inches thick and was placed on 2 inch pipe which carried sufficient current to heat it to a temperature corresponding to 160 lbs. pressure. One covering gave an efficiency of 84.2%, and the other 83.3%.

Mr. G. M. Brill reports a test of a magnesia covering placed on an 8 inch pipe. The pipe was subjected to a steam pressure of from 109 lbs. to 117 lbs. gauge, and the average difference in temperature was 272° Fr. The thickness and composition of the covering are not given. The test shows an efficiency of 85.81% over the bare pipe. When the differences in steam pressures and size of pipes are taken into account it is seen that the results given agree very closely with those given in Table 5.

The actual saving in dollars and cents for the different sizes of pipe with single and double cover is shown in Table No. 6. In figuring these results, several assumptions were made; 1st, that the steam was dry and saturated; 2nd, that the air surrounding the pipe averaged 70° F; 3rd, that the efficiency of the boiler was 60 per cent; 4th, that the cost of coal delivered at the plant was two dollars per ton; 5th, that the coal gave a calorimetric test of 12000 B. T. U.; and 6th, that the pipe was under pressure for 24 hours a day and 365 days in a year.

The cost of the covering as given includes the cost of applying it to the pipe.

The columns of this table show the loss by radiation per 100 lineal feet of the pipe per year, the loss for interest, depreciation of covering, etc., at 15% per annum, and the saving due to the use of the covering. It is clear from an inspection of this table that, in every case so far as tested, the single covering is more enomical than the double.

From the fact that the efficiency of the single covering gradually increases with the increased size of pipe, while the difference of efficiency between the single and double covering actually becomes less, it seems safe to say that for any size of pipe in ordinary use, say up to 8-inch, the double covering will not pay as well as the single.

1 INCH PIPE

TABLE NO. 1

FORM OF COVERING	Pressure	Length of Run. Min.	Difference in Temp. of Steam and Air	Dry Steam Condensed - Lbs.		B. T. U. Transmitted per Hour		Saving due to covering per hr. per sq. ft. of pipe		Saving in per cent	H.P. lost per 100 sq. ft. at 30 lbs per H.P. hour
				Per Hr.	Per Hr. per sq. ft. of Pipe	Per sq. ft of Pipe	Per sq. ft. of Pipe per Degree Diff. in Temp.	B.T.U.	Lbs. Dry Steam		
Bare Pipe..........	25.	180	194.20	3.545	.5755	532.49	2.7420				1.918
Bare Pipe..........	50.	180	208.86	4.552	.7394	668.97	3.2030				2.465
Bare Pipe..........	75.	180	234.26	5.360	.8702	773.05	3 3003				2 900
Bare Pipe..........	100.	180	246.82	5.824	.9455	839.18	3.3993				3.151
Single..................	25.	235	201.74	.9597	.1558	144.425	.7159	388.07	.4190	73.9	.519
Single..................	50.	240	210.71	1 0878	.1766	159.823	.7585	509.15	.5517	76.3	.588
Single..................	75.	240	235.67	1.3613	.2210	196.313	.8330	576.74	.6493	74.8	.733
Single..................	100.	240	250.87	1.5079	.2448	214.367	.8537	624.82	.7136	75.2	.816
Double................	25.	210	201.22	.7589	.1232	114.080	.5670	418.41	4514	79 3	.410
Double................	50.	210	219.93	.8291	.1346	121.824	.5540	547 09	.6047	82 7	.448
Double.......	75.	210	235.15	1.0589	.1719	152.729	.6495	620.32	.6983	80.4	.573
Double	100	210	256.04	1.0951	.1745	152.806	.5969	686.34	.7838	82 3	.582

3 INCH PIPE

TABLE NO. 2

FORM OF COVERING	Pressure	Length of Run. Min.	Difference in Temp. of Steam and Air	Dry Steam Condensed-Lbs.		B. T. U. Transmitted Per Hour		Saving Due to Covering Per Hr. Per Sq. Ft. of Pipe		Savings in Per Cent	H. P. lost per 100 sq. ft. at 30 lbs. per H. P. hour
				Per Hour	Per Hour Per Sq. Ft. of Pipe	Per Sq. Ft. of Pipe	Per Sq. Ft. of Pipe Per Degree Diff. in Temp.	B. T. U.	Lbs. Dry Steam		
Bare Pipe........	25.	120	194.67	8 833	.5142	476.55	2.448				1.7140
Bare Pipe.........	50.	120	203 97	10.204	.6033	545.82	2.676				2.0110
Bare Pipe...	75.	120	218 67	12.355	.7311	649.44	2.969				2.4370
Bare Pipe.........	100.	120	244.68	14.172	.8401	736.73	3.011				2.8003
Single	25	180	193.32	1.764	.1055	97 819	.5060	378.73	.4086	79.4	.3516
Single	50.	180	212.02	2 132	.1275	115.360	.5441	430.46	.4758	79.7	.4250
Single.................	75.	180	223.64	2.290	.1373	121.839	.5486	527.60	.6052	81.5	4576
Single.................	100.	180	248.61	2 625	.1570	137 630	.5536	599.10	.6838	81.6	.5233
Double	25.	210	203.25	1.275	.0763	70.690	.3478	405.86	.4379	85.8	.0425
Double	50.	210	207.93	1.466	.0877	79.366	.3817	466.46	.5155	86.4	.0488
Double	75.	180	230.04	1.627	.0973	86.311	3752	563.13	.6431	86.0	.0542
Double	100.	180	245.15	1.779	.1064	93.206	.3802	643.52	.7348	87.4	.0593

5 INCH PIPE

TABLE NO. 3

FORM OF COVERING	Pressure	Length of Run. Min.	Difference in Temp. of Steam and Air	Dry Steam Condensed - Lbs.		B. T. U. Transmitted per Hour		Saving due to covering per hr. per sq. ft. of pipe		Savings in per cent	H. P. lost per 100 sq. ft. at 30 lbs. per H. P. hour
				Per Hr.	Per Hr. per sq. ft. of Pipe	Per sq. ft. of. Pipe	Per sq. ft. of Pipe per Degree Diff. in Temp.	B. T. U.	Lbs. Dry Steam		
Bare Pipe..........	25.	120	187.24	12.584	.4717	437.20	2.335				1.572
Bare Pipe..........	50.	120	205.36	16.411	.6151	556.52	2.710				2 050
Bare Pipe..........	75.	120	209.63	17.800	.6672	592 62	2.837				2.224
Bare Pipe..........	100.	120	230.13	20 391	.7643	669.21	2.908				2.547
Single..............	25.	150	203.75	2.430	.0911	84 352	.4143	352.85	.3806	82.4	.304
Single..............	50.	150	216 03	2.809	.1053	95.312	.4412	461.21	.5100	83.8	.351
Single..............	75.	150	235.06	3.113	.1167	103.567	.4406	489.06	.5505	84.5	.389
Single..............	100.	150	245.29	3.097	.1161	101 624	.4143	567.59	.6482	85 6	.387
Double..............	25.	150	199.07	1.587	.0593	55.143	.2771	382.10	.4122	88.2	.198
Double..............	50.	150	213.90	1.848	.0693	62.736	.2933	493.79	.5458	89.2	.231
Double..............	75.	150	233.69	2.185	.0819	72.797	.3115	519.83	.5852	89.1	.273
Double..............	100.	150	253.85	2 282	.0855	74.871	.2951	594.34	.6788	89 9	.285

7 INCH PIPE

TABLE NO. 4

FORM OF COVERING	Pressure	Length of Run. Min.	Difference in Temp. of Steam and Air	Dry Steam Condensed - Lbs.		B. T. U. Transmitted per Hour		Saving due to covering per hr. per sq. ft. of pipe		Savings in per cent	H. P. lost per 100 sq. ft. at 30 lbs. per H. P. hour
				Per Hr.	Per Hr. per sq. ft. of Pipe	Per sq. ft. of Pipe	Per sq. ft. of Pipe per Degree Diff. in Temp.	B. T. U.	Lbs. Dry Steam		
Bare Pipe........	25.	90	173.60	14.669	.4346	402.75	2.320				1.448
Bare Pipe........	50	90	191.04	20.144	.5826	527 16	2.760				1.942
Bare Pipe........	75.	90	208.90	23.051	.6818	605 81	2.901				2.272
Bare Pipe........	100.	90	223 95	27.024	.7979	698.56	2.943				2.659
Single........	25.	120	200.30	2.4915	.0765	70.946	.3542	331 81	.3581	84.8	.255
Single........	50.	120	210.40	3.1532	.0931	84.160	.4000	443.00	.4895	85.5	.310
Single........	75.	120	230.90	3 5225	.1040	90 453	.4011	513.36	.5778	86.2	.347
Single........	100	120	248 46	3.5969	.1062	92.940	.3740	605.62	.6917	87.3	.354
Double........	25.	120	196 37	1.8168	.0536	49.689	.2537	353.07	.3810	89.1	.179
Double........	50.	120	212 63	2.0693	.0606	54.850	.2585	472.31	.5220	91.	.202
Double........	75.	120	235.92	2.3370	.0690	61.334	.2602	544.48	.6128	91.1	.230
Double........	100.	120	245.60	2.2663	.0671	58.944	.2400	640.62	.7308	91.8	.224

EFFICIENCY OF COVERINGS

TABLE NO. 5

NAME OF COVERING	Size of Covering Inches	Size of Pipe Inches	SAVINGS IN PER CENT					Cost of Covering Per Lineal Foot
			PRESSURE — LBS.				Average	
			25	50	75	100		
85% Magnesia Keasby & Mattison Co.	1	1	73.9	76.3	74.8	75.2	75.05	$ .18
“ “	1 and 3	1	79.3	82.7	80.4	82.3	81.2	.48
“ “	3	3	79.4	79.7	81.5	81.6	80.6	.30
“ “	3 and 5	3	85.8	86.4	86.	87.4	86.4	.76
“ “	5	5	82.4	83.8	84.5	85.6	84.1	.46
“ “	5 and 7	5	88.2	89.2	89.1	89.9	89.1	1.01
“ “	7	7	84 8	85.5	86.2	87.3	85.9	.55
“ “	7 and 9	7	89.1	91.	91.1	91.8	90.8	1.20

ECONOMY OF COVERING

TABLE NO. 6

FORM OF COVERING	Size of Pipe Inches	25 POUNDS			50 POUNDS			75 POUNDS			100 POUNDS		
		Loss		Saving	Loss		Saving	Loss		Saving	Loss		Saving
		Radiation	Interest Etc.		Radiation	Interest Etc.		Radiation	Interest Etc.		Radiation	Interest Etc.	
Bare Pipe	1	22.63			30.50			34.70			38.20		
Single	1	5.90	2.70	14.03	7.22	2.70	20.58	8.73	2.70	23.27	9.56	2.70	25.94
Double	1	4.68	7.20	10.75	5.28	7.20	18.02	6.81	7.20	20.69	6.69	7.20	24.31
Bare Pipe	3	53.75			67.90			82.75			89.98		
Single	3	11.10	4.50	38.15	13.80	4.50	49.60	15.30	4.50	62.95	16.50	4.50	68.98
Double	3	7.63	11.40	34.72	9.66	11.40	45.84	10.47	11.40	60.88	11.34	11.40	67.24
Bare Pipe	5	81.45			109.15			125.40			137.50		
Single	5	14.44	6.90	60.11	17.77	6.90	84.48	19.53	6.90	98.97	19.63	6.90	110.97
Double	5	9.66	15.15	56.64	11.82	15.15	82.18	13.80	15.15	96.45	13.98	15.15	108.37
Bare Pipe	7	111.00			152.90			176.00			191.20		
Single	7	16.90	8.25	85.85	22.70	8.25	121.95	24.35	8.25	143.40	24.30	8.25	168.65
Double	7	12.10	18.00	81.90	14.60	18.00	120.30	15.75	18.00	143.25	15.80	18.00	168.40

NOTE—This table is figured in Dollars per 100 lineal feet per year.

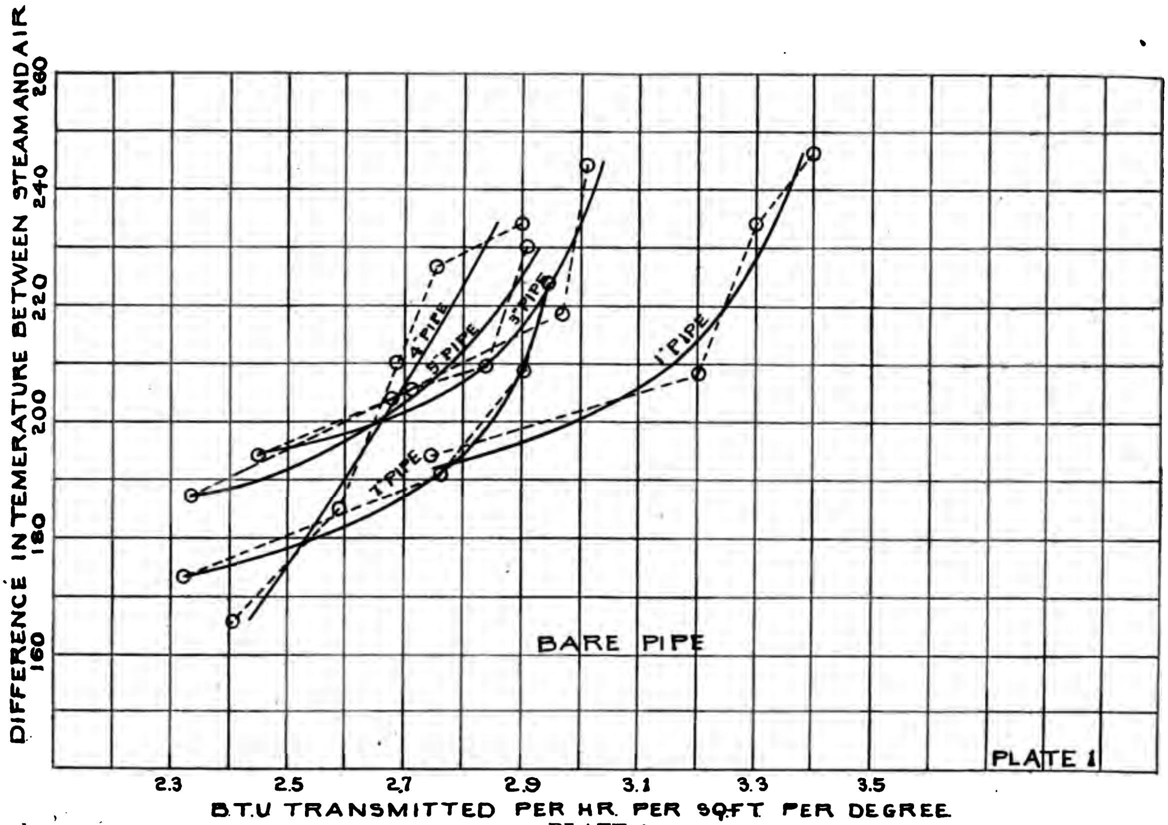
DIFFERENCE IN TEMERATURE BETWEEN STEAM AND AIR
160
180
200
220
240
260
2.3
2.5
2.7
2.9
3.1
3.3
3.5
B.T.U TRANSMITTED PER HR. PER SQ.FT. PER DEGREE
4" PIPE
5" PIPE
3" PIPE
1" PIPE
7" PIPE
BARE PIPE
PLATE 1

PLATE 1

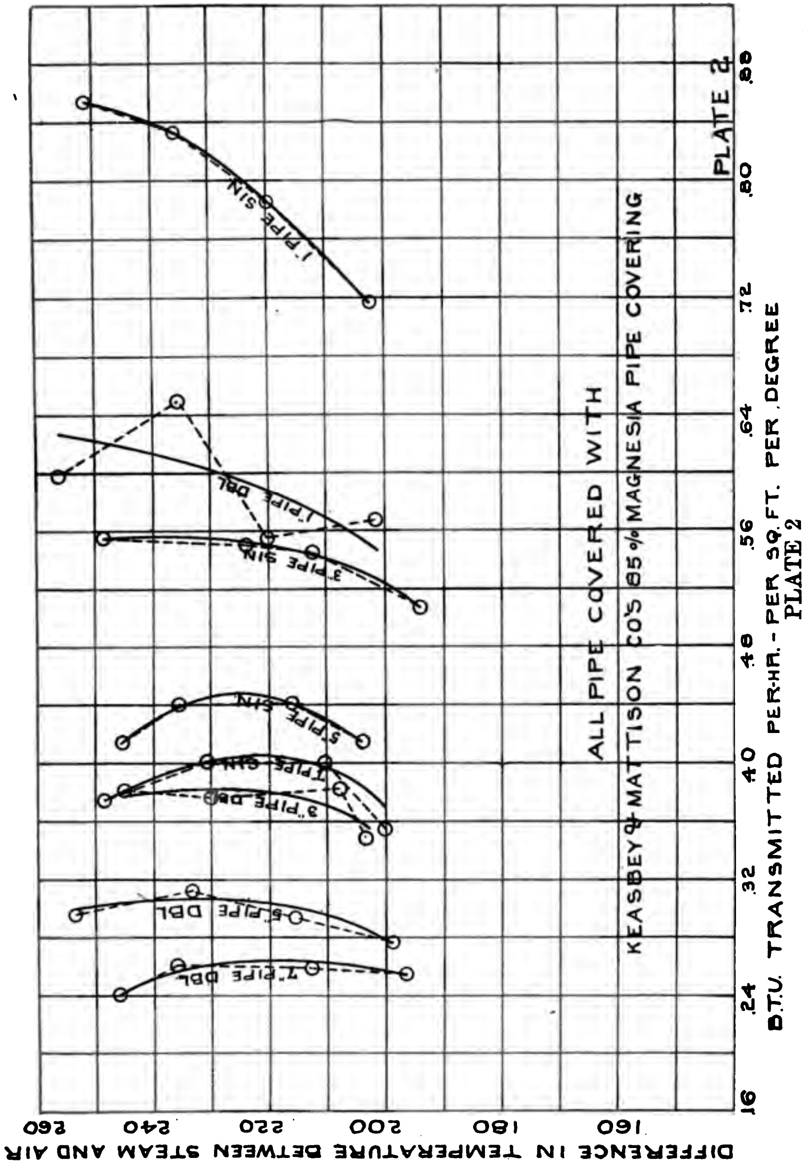
PLATE 2
ALL PIPE COVERED WITH
KEASBEY & MATTISON CO'S 85% MAGNESIA PIPE COVERING
1" PIPE SIN
1" PIPE DBL
3" PIPE SIN
5" PIPE SIN
7" PIPE SIN
3" PIPE DBL
5" PIPE DBL
7" PIPE DBL
.16
.24
.32
.40
.48
.56
.64
.72
.80
.88
B.T.U. TRANSMITTED PER HR. - PER SQ. FT. PER DEGREE
160
180
200
220
240
260
DIFFERENCE IN TEMPERATURE BETWEEN STEAM AND AIR

PLATE 2

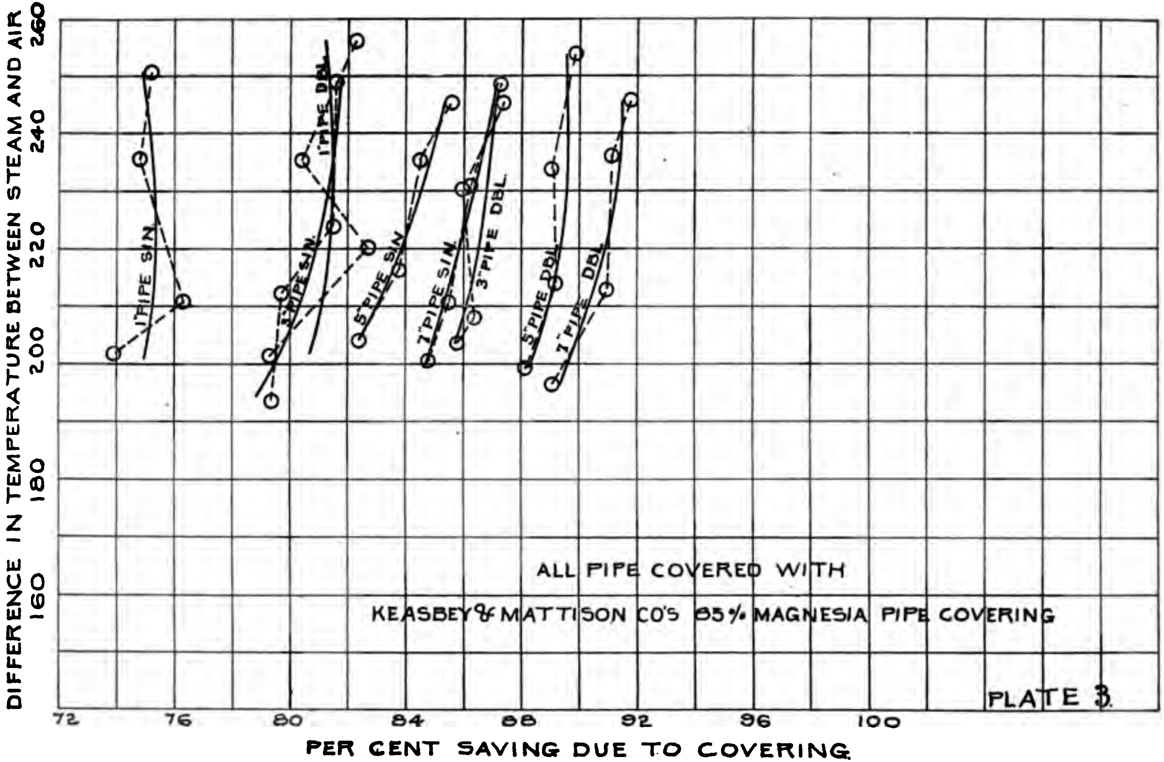
DIFFERENCE IN TEMPERATURE BETWEEN STEAM AND AIR
160
180
200
220
240
260
72
76
80
84
88
92
96
100
PER CENT SAVING DUE TO COVERING
1"PIPE SIN.
3"PIPE SIN.
1"PIPE DBL.
5"PIPE SIN.
7"PIPE SIN.
3"PIPE DBL.
5"PIPE DBL.
7"PIPE DBL.
ALL PIPE COVERED WITH
KEASBEY & MATTISON CO'S 85% MAGNESIA PIPE COVERING
PLATE 3.

PLATE 3

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The Assessment of Drainage Districts

BY

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The Assessment of Drainage Districts

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The increasing interest in the drainage of farm lands and especially the construction of the large drainage ditches and of tile drains under county control has caused this subject of assessments to be of much importance. It is therefore proposed in this outline to suggest certain principles which may govern the distribution of expenses over the various tracts, as well as to give an arithmetical method for actual solution of the problem. The general principle to be followed is that which is common by statute to all states, namely: that *assessments must be proportionate to the benefits received.* It therefore devolves upon the commission of assessors to determine proportionate benefits on the various tracts of land within the drainage district; also upon highways, railways and similar property.

The laws of Iowa (and of some other states) demand that the report of the assessors to the Board of Supervisors shall give a classification percentage which shall be 100 per cent on the tract of land which receives the greatest benefit, and proportionate percentages on all other tracts. The object of this percentage classification is held to be not alone a means for determining the present assessments, but of more importance for future use in case of an additional assessment which must be levied over the entire district for some purpose which affects the entire district in the same way as does the original construction. In case a new assessment must be made which does not thus affect the entire district, the statute expressly provides that the board of supervisors shall have power to cause a new classification percentage to be made. By the method to be here given the amount of assessment on the various tracts will be found, and that tract which has the largest assessment will then be marked "100 per cent" and other tracts proportionately.

Common Factors

While there are many factors which have a part in determining the amount of each assessment, the following three factors are common to all districts.

1st. If the drainage improvement is so designed that either by present construction or by future extension all of the wet land within the district will be reclaimed, then the land will be benefited in proportion to its intrinsic need for drainage, the lowest, wettest land being marked "100 per cent." (Table I, Col. 4.) In case the present improvement is not sufficient to drain all land properly, (e.

g. if the grade line is not low enough) then that land which receives the greatest benefit as regards changing the character of the land will be marked "100 per cent."

2nd. The land will be benefited in proportion to its proximity to the improvement, the land directly on the improvement receiving immediate benefit, therefore being marked "100 per cent," (Table I, Col. 5), while the land at some distance from the improvement will be marked a lower per cent.

3d. The land will be benefited in proportion to its distance from a *natural* outlet of its own, the land most distant from such *natural* outlet being marked "100 per cent" (Table I, Col. 6), while land near such outlet will usually be marked a low per cent as it receives the benefit of only a part of the improvement.

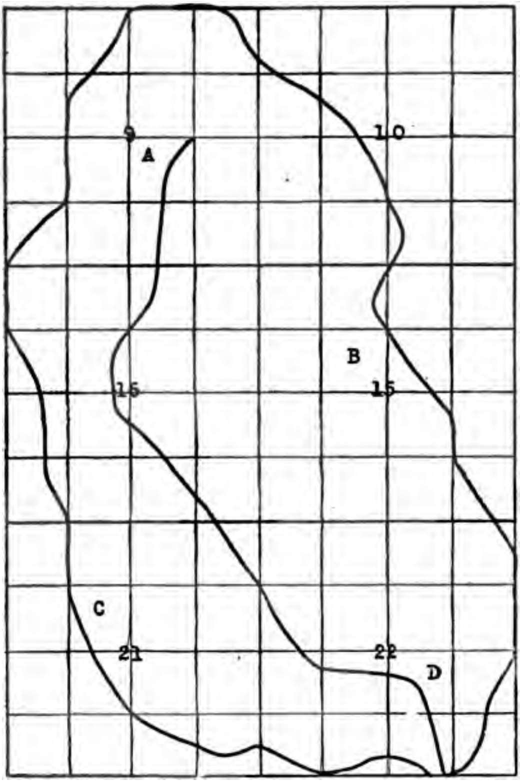

Fig. 1. DRAINAGE DISTRICT.

Solution for an Assumed District

TABLE I. ASSESSMENT OF A DRAINAGE DISTRICT.

1	2	3	4	5	6	7	8	9	10
Owner	Description	FACTORS				Product Factors	Sum for Tract	Assessment	Classification Per Cent
		Acres	Charact'r Land	PROXIMITY					
				Drain	Outlet				
A	N.W.-S.E. 9	6	100 or 3-3	100	100	6.00			100 if Maximum
		12	2-3	100	100	8.00			
		15	1-3	90	100	4.50			
		7	0						
		40					$18.50	$245.00	
B	S.E.-N.W. 15	6	3-3	10	65	.39			6.6
		12	2-3	10	65	.52			
		15	1-3	10	65	.32			
		7	0						
		40					1.23	16.00	
C	S.E.-N.W. 21	4	2-3	50	50	.67			8.1
		10	1-3	50	50	.83			
		18	0						
		32					1.50	20.00	
D	N.W.-S.E. 22	6	3-3	100	20	1.20			20.0
		12	2-3	100	20	1.60			
		15	1-3	90	20	.90			
		7	0						
		40					3.70	49.00	
							Etc.	Etc.	Etc.
							1280.40	$17000	

Referring to Figure 1, let us assume a drainge district with outlet at the south side of Section 22, the upper end of the drain being just east of the center of Section 9. Referring to Table I, it will be noted that column 1 gives the name of the owner, while column 2 gives the description of each tract, be it forty acres or less according to any natural or recognized subdivision. Columns 3 and 4 show the subdivision of each tract into various classes of land, in this case using four classes, namely: the very wet or slough land which will receive maximum of benefit, therefore marked "100 per cent or 3 / 3" in column 4; the "wet" land which is of such character that it will receive two-thirds as much benefit; the "low" land which will receive one-third as much benefit, and the "dry" land which is commonly rated to receive zero benefit. (A method for assessment of dry land due to general benefit will be given later.) For purpose of comparison let us assume that the tracts A. B. and D, each having forty acres, are similarly classified, each estimated to have six acres of the "100 per cent" or "3 / 3" land, twelve acres of the "2 / 3" land, fifteen acres of the "1 / 3" land, and seven acres of the "zero" land. Sometimes such factors as 90%, 80%, etc., will express the rating for a tract, and the fractions will not be used. Tract C on the outer edge of the drainage district has only thirty-two acres. As regards character of land and its intrinsic need for drainage, columns 3 and 4 give the required data.

Column 5 shows factors which represent proportionate benefits on account of proximity of drain. It will be easily recognized that tract B receives very little benefit from this improvement as compared with tracts A or D, therefore the first two classes of land in tract A are rated 100 per cent because the drain goes directly through such tracts. As a suggestion the fifteen acres of 1-3 land are rated at 90 per cent, it being sometimes noted that this low land lying back from the slough will require still further improvements in order to render its soil as good as will be that nearer to the improvement. A similar classification is made for tract D. Tract B lying a mile back from the drain is here given a rating of 10 per cent. This is suggestive only. It might perhaps be twenty per cent or as low as three per cent. Tract C lying at a lesser distance from the drain than tract B is given the suggestive rating of fifty per cent. The *natural* fall from outlying tracts toward the improvement may here be given proper credit, the factors being reduced as available fall increases. In column 5 is then given the estimated comparative ratings due to the benefits which the various tracts will receive on account of proximity to such improvement.

There may seem to be at this point a conflict with the Statutes of Iowa which provide that we "shall not consider what benefits such lands will receive after some other improvements shall have been constructed, but only the benefits which will be received by reason of the construction of the improvement in question as it affords an outlet for the drainage of such lands." The second clause of this quotation must be recognized as fully as the first. It is not proposed to assess land lying back from the improvement for anything more than the benefit it derives on account of an outlet which is brought to a nearer point than the former outlet. Land remote from the improvement is benefited because of its new and more accessible outlet, though the actual condition of the soil may not be changed in any respect until further extension of the improvement is made. In the same way is farm land benefited when a new railroad brings its warehouses and markets to within a short distance of said farm where before such privileges were a long way distant. The character of the soil and the farm improvements are not changed in any respect, but the farm becomes more valuable because of a more accessible market and distributing point. In this way are all parts of the drainage district benefited, even though remote from the present improvement.

Column six gives the comparative ratings for benefits on account of outlet. If this district has a gradual fall from the upper end to its outlet with no sudden slopes or hills, then will the tracts at the upper end receive the maximum of benefit on account of outlet. The tract next to the outlet receives benefit from the improvement above in so far as overflow is prevented. On many of our districts the tract at the lower end receives very little benefit from the improvement above. An outlet to a tile drain may be of great benefit as it may furnish running water for stock throughout the entire year. As a suggestive figure, tract D is here (column 6) rated at 20 per cent, tract C 50 per cent, tract B 65 per cent, and tract A 100 per cent on account of benefit of outlet. If tract A had a good outlet of its own, for example, on to the tract just below it, then A would be assessed a small amount because it would receive little benefit from the construction of the drain below, and the tract just south of A would then perhaps become the "100 per cent tract" on account of the outlet. The principle is simply this, that that tract of land which will receive the greatest benefit on account of an outlet, taking into account the distance from a natural outlet and the length of improvement necessary for drainage, will be rated at 100 per cent.

Columns 3, 4, 5 and 6, therefore, give the various factors which are common to most drainage districts. On the basis of an assessment of one dollar per acre on that land most benefited, the 6 acres in tract A would be assessed $6.00 (column 7) this result being obtained by multiplying together the factors. The 12 acres of tract A would be assessed $8.00, the 15 acres $4.50, making a total on this one dollar basis of $18.50 against tract A. As a comparison, tract B, having similar acreage but located a long distance from the drain and not so far from the outlet, would be assessed $1.23. Tract D, having similar acreage located on the drain but near the outlet, would be assessed $3.70. By similar figures C would be assessed $1.50. Taking all the tracts of the drainage district, let us assume that the sum available on this one dollar basis is $1280.40. Let us assume too that the estimated cost of the improvement, less the lump sum assessments for railways, highways, etc., is $17000.00. Then will column 9 give the actual assessment against each tract, these figures being found from column 8 by using the ratio between the summation figures at bottom of columns. It is sufficient to make these final calculations to the nearest dollar.

As before stated the statutes of Iowa require a classification percentage. Therefore, by noting neither in column 8 or column 9 the maximum amount, place opposite that amount in column 10 the figure "100" assumed in this table to be for tract A. On this basis tract B would then be rated at 6.6 per cent, tract C at 8.1 per cent, while tract D would be assessed 20.0 per cent. This classification percentage is held to be made with respect to the amount of the assessment but without respect to the magnitude of the tract. If for any purpose it is desired to make comparative rating per acre, then will the 8.1 per cent for tract C be increased in the ratio of 40 to 32, making 10.1 per cent, there being only 32 acres in this tract as compared with 40 acres in the others.

Columns 9 and 10 are very readily obtained from column 8 by use of a slide rule, with which every engineer on assessment commissions should be equipped.

Other Factors

On a drainage district it is possible that some tracts of farm land are located near a town, while others are at a great distance. The value of the land near market is, on account of such market, relatively higher than that at a distance. It may, therefore, be held that the drainage of this land near market is of more benefit proportionally than is the drainage of similar land in more remote parts of the district. If it is desired to take this factor into account, another column can easily be added to the table here given, placing such between columns 6 and 7, and giving therein such percentages as will express proportionate rating of benefits due to comparative value of land and benefits to be received. This can be extended for other factors.

When a drainage district includes a city or portion thereof, the assessment of city lots will then require careful consideration. By some method the benefit to the city lots must be determined and compared with the benefit to the farm property. The principle of assessment may thus be stated. If a city lot is benefited two hundred dollars while a forty acre tract of land is also benefited two hundred dollars, then the assessment of these two tracts of land must be equal. This is independent of the areas of the tract. The general principle must be remembered that *assessments must be proportional to benefits received.*

A special case which may occur is the following: The lower part of the improvement may be an open ditch while the upper part may be a large tile drain. That tract just above the outlet of the tile will receive greater benefit because of the tile than it would if an open ditch were here constructed; therefore the assessment on this tract would be proportionately larger than on the others.

If a tract has been already tiled out and the outlet to the tile will be into the new improvement, then should the tract be assessed for the benefit which it will receive on account of the new and better outlet.

In the improvement of rivers to prevent overflow, some scheme of classification and calculation as here proposed may be used, but particular attention must be given to the value of factors, especially the outlet factor which is liable to be more nearly a constant quantity rather than such a variable as is given for the district in Figure 1.

The Assessment of Dry Land

It is often considered advisable to levy a small assessment on all land within the Drainage District on account of the general benefit, because such improvement is conducive to the public health and general welfare of the entire district, and furthermore the irregular tracts of dry land which may be cultivated will be made square and regular as soon as the proper drainage improvements have been constructed. It is true that much of the so-called dry land would be benefited by tiling and such improvements will doubtless be made in the not distant future. However, our farmers are not willing to admit such possible benefits at the present time and assessors are therefore reluctant to levy such assessments. Some engineers may prefer to assess the dry land a small amount at the time that the assessments on other classes of land are made as in Table I. It is here proposed, however, that an assessment be levied over the entire district. Assume such assessment to be 10 cents to 50 cents per acre, depending on the total area of the district and total cost of the improvement. To the amounts of assessment as given in Table I will then be added the assessment here made for general benefit, this assessment being made on the wet land as well as on the dry land.

Assessment of Highways

This is perhaps the most puzzling question which occurs on the assessment of a drainage district. It is probable that such assessments are usually only a guess. While no definite rules can be given, some principles may be noted. The assessment should vary with the importance of the road. The road which is a main highway leading to an important market will receive greater benefit from drainage of its roadbed than will an unimportant highway which now is or will later be but little used because of its location. An estimated cash benefit may be computed taking into account the present expense of maintaining the highway after drainage; also the cost of construction of a suitable highway both before and after the construction of the drainage improvement. These maintenance charges expended annually should be capitalized, thus giving the proper sum of money which at an ordinary rate of interest will yield the amount necessary to provide for maintenance. The actual cash benefit to some tract of farm property may then be estimated and comparison taken with the benefit which the highway will receive. From these comparative benefits, and noting the assessment on the tract of farm land, the assessment on the highway may be obtained. The cost and maintenance charges of bridges and culverts, both in present condition and after construction of the improvement, must be estimated. The comparative figures will furnish data for determining benefits and also damages.

It does not seem just to assess highways by the acre at the same rate as adjoining farm land. The benefit to the highway does not depend upon the area of the highway. The part of the highway actually used is the 16 or 20 foot width of driveway. It is entirely immaterial whether the highway be four rods wide or half that amount or double that amount. The benefit which the public will receive in using the highway depends upon the condition of the embankment and not upon the distance between the fence lines.

Assessment of Railways

As in the case of highways, there are many factors to be considered, some of which may be only rough estimates. It is held by some that, even if the roadbed and bridges of the railway company may not receive any benefit, still the railway company is benefited because of the drainage improvement over the district which will serve to give increased crops, the transportation of which is the business of the railway. The drainage of a roadbed is certainly of benefit to the railway company, as it renders such roadbed more stable and consequently will require a less annual expense for maintenance. Very frequently the railway company will be able to use a much shorter bridge after the construction of the improvement. This is a factor of great benefit. As in the case of highways, the difference in cost of maintenance of the present and of the future structures and the cost of construction thereof must be estimated, the annual charges being capitalized.

These assessments against highways, railways and any general assessment over the entire district are all lump sums which will be subtracted from the total cost of the improvement, the remainder being then distributed over the tracts of wet land, as shown in Table I. (See page 17).

Mode of Procedure

The method of using the system here proposed may be outlined as follows: Before going into the field the engineer reviews the plats of the drainage district and in his note book makes out an outline for the tabulation of Table I. He records the description of each 40-acre tract (column 2) and the owner thereof (column 1). Going into the field the commissioners review each 40-acre tract and determine the areas of different classes of land (columns 3 and 4). The only work done in the field is to determine this classification for character of land. The classification for proximity to drain and outlet is determined after the return to the office. It is often impossible to make a comparative rating for these factors while in the field. This can be very readily accomplished in the office after the field investigations by use of the map, from which may be noted the comparative distances, watershed divides and other topography. The factors thus determined are multiplied together and total assessments calculated.

THE DISTRIBUTION OF EXPENSES TO SUBDISTRICTS

Referring to Fig. 2, and Table II, let us assume that we have a drainage district in which there are laterals as shown. In the figure, data are given for each sub-district. For example, sub-district A has 900 acres; the cost of the improvement within sub-district A will be $2,500.00. (F. 80 refers to a factor to be explained later). As a general principle, but having its exceptions, each sub-district, as A, B or D, should be assessed for ts own improvements within its borders. This will hold unless it can be shown that the improvement in some sub-district, as A, will be of benefit to some other sub-district, as B or C. The drain through

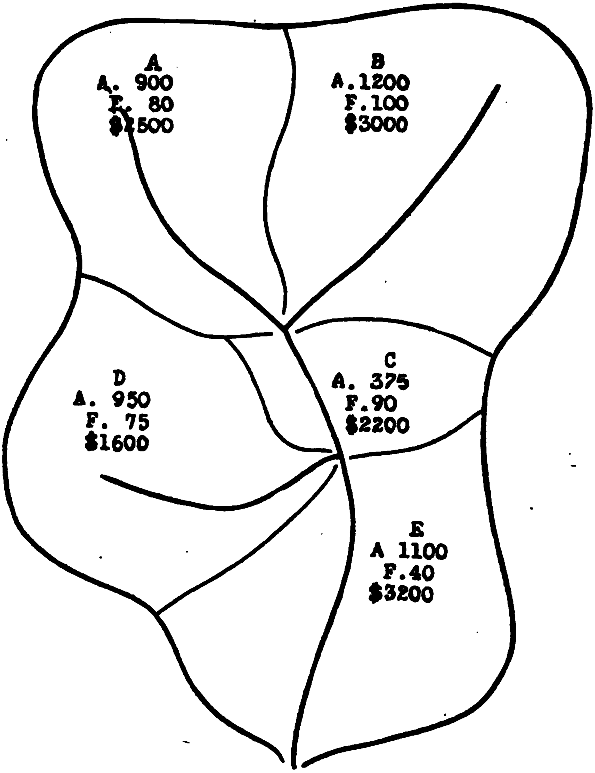

Fig. 2. SUB-DISTRICTS.

sub-district C, however, is of service not only to C but also to A and B, therefore, the cost thereof ($2,200.00 for C) should be distributed in proportion to benefits received. As in Fig. 1, let us assume that an acre of land at the lower edge of tract C receives but little benefit from the improvement above, and that an acre of land at the upper edge of tract C receives a maximum of benefit. If the change were gradual from zero to 100 per cent, the average benefit to tract C would then be 50 per cent. In Table II, however, it is assumed that the average benefit is two-thirds instead of one-half. On the basis of acreage take two-thirds of the 375 acres in sub-district C and combine this with the acreage of sub-districts A and B and divide the expenses ($2,200.00) through C proportionately. This is shown in the first part of Table II under the heading, "Basis of acreage." Similarly the expense of improvement through E ($3,200.00) would be divided over A, B, C, D and E, using 2-3 of 1,100 acres in E to combine with the other acreages. In this way will be found the various distributions and in the lower part of Table II is given the total expense to each sub-district. For example, sub-district A is assessed $2,500.00 for its own improvement, $842.00 for its outlet through C, $693.00 for its outlet through E, a total of $4,035.00.

TABLE II.—SUB-DISTRICTS

For Part	Assess to	Basis of Acreage		Basis of Factors			
		Acres	Assess	Acres	Factor	Equiv. Area	Assess
C	A	900	842	900	80	720	738
	B	1200	1124	1200	100	1200	1230
	2-3 C	250	234	250	90	225	232
		2350	2200	2350		2145	2200
E	A	900	693	900	80	720	706
	B	1200	924	1200	100	1200	1177
	C	375	288	375	90	338	332
	D	950	731	950	75	712	698
	2-3 E	733	564	733	40	293	287
		4158	3200	4158		3263	3200

TOTALS

To District	For Part	By Acreage		By Factors	
A	A	$2500		$2500	
	C	842		738	
	E	693	$4035	706	$3944
B	B	3000		3000	
	C	1124		1230	
	E	924	5048	1177	5407
C	C	234		232	
	E	288	522	332	564
D	D	1600		1600	
	E	731	2331	698	2298
D	E	564	564	287	287

The preceding distribution has been made on the basis of acreage, but let it be assumed that sub-district B contains much more wet land on the average than does sub-district A, then should B pay more proportionately than does A. By an inspection of the various districts it may be possible to determine factors which will represent proportionately the different conditions of the sub-districts. This can be much more accurately determined, however, by a review of columns 3 and 4, Table I, which columns give the acreages for different classes of land.

Let it be assumed that sub-district B averages more wet land than any other, therefore, rate this subdistrict as 100 per cent. Assume sub-district A to rate 80 per cent, C 90 per cent, D 75 per cent, and E 40 per cent. Instead of distributing the expense over the various sub-districts in proportion to the acreage, we may allow for this difference in character of land by multiplying the various areas by their respective percentages, which will give equivalent comparative areas. By a method of procedure similar to that used on the "basis of acreage," distribute the expense, using these new equivalent areas. Table II shows that B would then be assessed $5,407.00 on the basis of factors and only $5,048.00 on the basis of acreage, the factor basis taking account of the wet character of the land.

A very common case in connection with construction of a drainage improvement is that not all of the laterals are put in at time of first construction, but the main ditch is properly designed to provide for all future laterals. Assuming that the lateral into sub-district A is not constructed, the proper assessment against sub-district A should then be its proportionate share of the expense through C and E, being $842.00 plus $693.00 or $1,535.00 on the acreage basis, or $1,444.00 on the factor basis.

The distribution of total cost to each sub-district over the various tracts within such sub-districts is important. Considering the amount $4,035.00, obtained for sub-district A on the acreage basis, the first part, $2,500.00, should be distributed by the method of Table I, using ratings for proximity to outlet as there shown, the outlet to sub-district being considered. The other parts, $1,535.00, being cost of construction through C and E, should be distributed by the same method, Table I, but using a factor 100 per cent for all tracts as regards outlet, thus giving column 6 a constant value. The work may be made much more simple and of sufficient accuracy by the following method, using an example from Table II:

For A, the cost of construction through C and E is $1,535.00, or 38 per cent of the total, $4,035.00. Mark that tract of land 38 per cent (Col. 6), which receives zero benefit on account of outlet due to the construction of the improvement through A. Mark the tract most benefited 100 per cent, and to all others give proportionate values between 38 and 100 per cent. If there is to be considered no tract of zero benefit then will the tract least benefited be rated something above 38 per cent and proportionately between 38 and 100 per cent, depending on the estimated rating due to the part through A.

APPRAISEMENT OF DAMAGES

Our statutes insist that the commission which assesses damages shall give no thought whatever to any benefits, and that the commission which assesses the benefits shall not think of the damages. While the subject of this paper is the assessment of benefits, it may not be inappropriate to mention a few factors which may be considered as causes for damages.

A tile drain across a field is of damage only in so far as does the construction of such tile drain interfere with the crops and farming operations. An open ditch, however, is an obstruction for all time, and, like the right of way of a railway company, may be considered just cause for damages. It is held by some that land which is of little present value is, if used for a drainage ditch, subject to award of damages only for such present value. However, the assessors of benefits will assess the land for the benefit that it will receive without regard to the fact that a certain area thereof is to be used for an open ditch. Therefore should the land be paid for at the rate of value which the land will have after construction of such ditch. This may be further illustrated as follows: Supposing that a property line between owners A and B runs along the middle of a draw or slough. If the ditch is located entirely on A's land then B will get the full benefit of the ditch without giving up any of his land, whereas if the ditch is located entirely on B, A will then receive the full benefit without loss of land. Similarly do parties owning land back from the ditch get the benefit of the ditch as an outlet without giving up their land for such ditch. For this reason should the right of way taken for the ditch be purchased, the rate of payment being that of the value of the land after improvement has taken effect.

There are other damages due to an open ditch. The ditch itself cutting across fields, perhaps diagonally, is an obstruction for farm operations for all time to come. Waste banks are unsightly and are a possible injury to the adjoining fields. Extra fencing must be put up to keep stock from the ditch. All of these factors are to be considered in estimating the damage due to construction of an open ditch across a farm.

In awarding damages to a railway company many factors are to be considered and these are often as indefinite as are those for the assessment of benefits against such companies. If the company is required to build a new bridge there will be damages on this account. If the company will be required to take up a bridge to allow the passage of the ditching machine there will be a just claim for damages. If the company is required to pay the expenses of constructing the improvement across its own right of way, such cost will be a just claim for damages. In few cases, however, does it seem necessary to put in a steel bridge of long span to replace a present timber structure of several short spans, this change to be made at the expense of the district. By making the ditch a little wider at the point of crossing the railway right of way pile bents may often be put into the channel of the ditch, thus permitting the use of a pile trestle similar to what may now be in use. (See page 17).

It will be noted that benefits and damages are in many cases very intimately connected. Damages are awarded because of a change of the railway bridge and benefits are also assessed against the same company for the same bridge, because the new bridge will be, perhaps, shorter than the present bridge. It therefore seems advisable that part of the members of one commission should also be on the other commission, and there are many reasons why it would be advisable to have the same entire commission appraise the damages and assess the benefits.

THE SURVEY, MAP AND REPORT

Methods of Surveys for Drainage Improvements

The method of making the investigation of the drainage district relative to a proposed improvement will depend entirely upon the physical characteristics of the district, its extent, the degrees of efficiency of drainage desired as regards whether only a main drain shall be constructed or whether sufficient laterals shall also be constructed in order to give an outlet to every farm within the district, etc. On small districts and perhaps on some of the larger ones the ordinary method of surveying by tape and transit may be best used. Sometimes levels are taken at the same time, using the transit as a level. On many districts the stadia method of surveying is desirable. As it will not be necessary to use vertical angles the stadia methods are, therefore, very much simplified. This gives a very easy method for locating the boundaries of sloughs and for getting such topography as is necessary. The engineer's level is then used for getting the required elevations. On large districts the "intersection method" may be economically used. By this method a base line is laid off, using either the tape or stadia, and a triangulation system is developed to cover the entire district. From these triangulation points stadia work may be projected or intersections taken to flags (cloth on lath) to give fence corners, boundaries of sloughs, draws, etc. This method has been developed in considerable detail and explanations may be found in a paper by the author given in the Journal of the Western Society of Engineers, Vol. 10, page 204, April, 1905, and reprinted in the Engineering News, Vol. 53, page 329.

It is not considered necessary to make a survey by level, taking elevations every 100 feet along any certain line, but it is essential to have elevations of important points over the entire drainage district. These points should include the water level of all ponds and depths of same, levels at strategic points along the natural channels, also lowest points at divides between ponds, draws, etc. Most of these elevations can be most profitably taken after the map has been prepared, because the map will show the real location of various points and the level will be used to give elevations to show possibilities of constructing the improvement along certain routes, keeping in mind cut-offs and other short routes, many of which would not be noted in the field, but will be apparent only by reference to the map. If the main ditch only is at first constructed it must be so designed that it serve outlying districts by construction of future laterals. Otherwise the ditch may need to be made larger and deeper for such new improvement. The method of designing improvements so that every farm within the district is given an outlet is to be commended.

Map

It may be said that one main object of the survey is the preparation of a map from which the proposed system may be designed. The survey and the map are, therefore, very intimately connected. The map should be carefully made to scale and should show the entire watershed which is by nature tributary to the proposed improvement; also the divides between the various sub-districts, as it is impossible to design the improvement without such data. Elevations and depths of ponds should be shown, also elevations of the low necks of lands and of all important points of land along the natural channels. A paper location may then be made by the engineer and such a profile platted as will enable him to design the system. This map will also be of service and usually sufficient in purpose for use of the commission which assesses the benefits, though in some cases it may be advisable to have the engineer make the location survey after the district is established and before the assessment is made, the first survey being preliminary only.

Report to the Board of Supervisors

The engineer's report should be of such a nature as will show the advisability of establishing the district and constructing the proposed improvement. In addition to the map and a report as to the number of acres within the district and the estimated cost thereof, there should also be some data furnished which will enable the board of supervisors to get an idea as to the cost per acre which will be assessed against various classes of land. In table III, here submitted and which is used in many counties to advantage, the number of acres of various classes of land is given, also an estimated average assessment from which may be determined the revenue available, or more properly the average assessment is determined which will yield the sum required for the construction of the ditch.

TABLE III.—ESTIMATE FOR PROPOSED IMPROVEMENT

TRACT	ACRES DIRECTLY AFFECTED			ACRES INDIRECTLY AFFECTED			
	Swamp	Wet	Low	Swamp	Wet	Low	Dry
N. W. 9............	30	20	30		5	10	65
E. ½ N. E. 9.......	12	20	10			10	28
S. E. 8............		7	10	20			
	....	....	Etc.	....	Etc.	....	....
Totals	210	390	540	68	240	760	1320
Assessment	$ 8.00	$5.40	$2.70	$1.50	$1.00	$ .50	
Revenue Available...	$16.80	$2110	$1460	$102	$240	$380	

$6072 available on 2208 Acres, also 1320 Acres of dry land.
Estimated cost of ditch $6000.

In the table it is noted that $8.00 per acre is the average assessment against swamp land directly affected by the improvement. It is probable then that the maximum assessment will be less than double this amount, or $16.00. From such a table may be determined at once the advisability of constructing the improvement. Acreage data of sufficient accuracy for this report can be readily made up by the engineer after his field investigations.

DAMAGES TO A RAILWAY COMPANY

Since the preparation of the paper on "The Assessment of Drainage Districts," attention has been called to the decision of the Supreme Court of the United States, affirming the Illinois decision in the case of the C. B. & Q. Ry. Co. vs. Commissioners of Drainage District No. 1, Bristol Twp., Kendall Co., Ill. (212 Ill., 103). This decision is of great interest in dealing with award of damages to railway company on account of a new bridge which may be required to cross a drainage improvement along a natural waterway. If the statutes of other states are not already so framed as to take advantage of this decision, it is probable that proper amendments will soon be made.

The following quotations taken from an interesting context may serve to give the gist of the matter:

"When the railway company laid the foundations of its bridge in Rob Roy Creek it did so subject to the rights of the public in the use of that watercourse, and also subject to the possibility that new circumstances and future public necessities might, in the judgment of the state, reasonably require a material change in the methods used in crossing the creek with cars. It may be—and we take it to be true—that the opening under the bridge as originally constructed was sufficient to pass all the water then or now flowing through the creek. But the duty of the company, implied in law, was to maintain an opening under the bridge that would be adequate and effectual for such an increase in the volume of water as might result from lawful, reasonable regulations established by appropriate public authority from time to time for the drainage of lands on either side of the creek." * * * "The great weight of authority is, that where there is a natural waterway, or where a highway already exists and is crossed by a railroad company under its general license to build a railroad, and without any specific grant by the legislative authority to obstruct the highway or waterway, the railway company is bound to make and keep its crossing, at its own expense, in such condition as shall meet all the reasonable requirements of the public as the changed conditions and increased use may demand." * * * "The duty of a railroad to restore a stream or highway which is crossed by the line of its road is a continuing duty; and if, by the increase of population or other causes, the crossing becomes inadequate to meet the new and altered conditions of the country, it is the duty of the railroad to make such alterations as will meet the present needs of the public." * * * "The duty of the company will end when it removes the obstructions which it has placed in the way of enlarging, deepening and widening the channel. It follows, upon principles of justice, that while the expense attendant upon the removal of the present bridge and culvert and the timbers and stones placed by the company in the creek, as well as the expense of the erection of any new bridge which the company may elect to construct in order to conform to the plan of the Commissioners, should be borne by the railway company, the expense attendant merely upon the removal of soil in order to enlarge, deepen and widen the channel must be borne by the District."

Engineering Experiment Station

IOWA STATE COLLEGE

Ames, Iowa

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Tests of Iowa Limes

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TESTS OF IOWA LIMES.

BY IRA A. WILLIAMS

GENERAL CONSIDERATIONS

The lime of commerce is produced by the calcination of limestone and varies in composition and purity therefore as do the limestones themselves. The latter range from practically pure calcium carbonate ($CaCO_3$) to the sandy and clayey limestones in which the impurities compose a large percentage of the rock. Again, the calcium may be in part replaced by magnesium which gives the magnesian limestones, and if this replacement has taken place to the extent that magnesia (MgO) comprises eighteen to twenty per cent of the stone, the term dolomitic limestone is more commonly applied.

A limestone composed essentially of $CaCO_3$ will furnish a high grade of quicklime, one containing little else than CaO; one composed of $CaCO_3$ with a greater or less percentage of $MgCO_3$ will afford a magnesian or dolomitic lime; while the argillaceous limestones will give a product of a degree of purity depending on the amount of clay in the original stone. The properties of the resulting limes will vary according to their composition.

Limestones are widely distributed in nature, both geographically and geologically. They are found interbedded with and overlapping other common sedimentary strata, and they have been produced in much the same way as other sedimentary rocks. Good reasons are readily conceived why they should be apt to partake of the nature of, and to grade into or be contaminated with other sedimentary materials. It is, nevertheless, not at all uncommon to find limestones that run over ninety per cent lime carbonate, and occasionally as high as ninety-eight or ninety-nine per cent. The analysis of nine samples of non-dolomitic Iowa limestones show a range of from 82.5 to 97.02 per cent carbonate of lime, three of the nine samples showing over ninety per cent.

As indicated, limestones depart in composition most commonly in the content of magnesia and in the clay and sand impurities. The effect of these substances on the resulting lime is of so much importance that they may be given separate consideration.

WHITE VERSUS BROWN LIMES.

A pure limestone when burned changes to calcium oxide (CaO) by the loss of carbon dioxide (CO_2) gas. The resulting lime is the white lime of commerce. It slakes readily and rapidly, with the evolution of much heat and becomes a perfectly white paste. The chemical reaction on slaking is:

Quicklime Water Slaked Lime

$$\text{CaO} + \text{H O} = \text{OCa}(\text{OH})_2 + \text{Heat}$$

This reaction with CaO furnishes with maximum intensity all of the evidences of the chemical change which is taking place. Any impurities in the quicklime, which may have been present in the limestone, tend to retard and to make less vigorous the slaking process, but the quantity of the impurity must be considerable before any pronounced interference will be noticeable.

Dolomitic limestones are very common and produce limes that slake slowly, evolve less heat and are of a gray or brown color. Over ten per cent of MgO must be present to appreciably alter the properties of a lime. Limes containing less than ten per cent MgO are accordingly spoken of as magnesian limes, while those over ten per cent are properly called dolomitic or brown limes. Dolomitic limes produced from Iowa limestones range in magnesia content from 15.23 to 35.73 per cent in the marketed product.

With reference to the amount of magnesia contained, therefore, limes may be classified; (a) hot, white or high-calcium limes when MgO is below ten per cent; and (b) cool, brown, or dolomitic limes when MgO is over ten per cent. The value, properties and adaptability of the two classes will be briefly treated later.

WHITE VERSUS ARGILLACEOUS AND SILICEOUS LIMES.

Limestones which contain clayey matter produce limes that are gray in color and less vigorous in their action than the pure white limes. Sand or other siliceous impurities in the lime do not ordinarily exert any influence on its physical properties, acting simply as a diluent in the same way that sand does in a mortar. Should the latter be excessively fine, however, it may become susceptible to chemical combination with the lime if high temperatures are employed in the burning. As the amount of sand and clay increases, limestones are called sandy or arenaceous and argillaceous, and when these materials become the predominant constituents, the rock passes into calcareous or limy sandstones and shales.

The presence of a small proportion of the above named impurities causes limes to become hydraulic, that is, to possess the property of hardening under water, by the gradual taking of water into chemical combination. When the quantity of clayey impurities in the limestone reaches 6 per cent, they begin to produce hydraulicity. Below 6 per cent the only noticeable effect is in the retardation of slaking. Limestones carrying six to twelve per cent furnish limes that are cool and slow slaking, gray in color and make the best of mortar if burned at ordinary lime kiln temperatures. There is, however, much greater danger of overburning than in the case of white limes as the clay has a tendency to combine with the lime, which decreases its value unless finely ground. Fifteen to twenty-five percent of clay renders a lime strongly hydraulic if properly burned, but care is required to avoid overburning and consequent clinkering. If clinkering is permitted to take place, the product is found to possess after fine grinding a hydraulicity greater than that of the hydraulic limes and similar to cement, which property becomes more prominent the greater the clay content and higher the temperature of burning. The limit at present is Portland cement in which the raw materials are artificially blended and thoroughly clinkered at very high temperatures.

It is clear, therefore, that limes are but one end of a series of products of which Portland cement is the other. The dividing lines separating the various members of the series that are put upon the market are to some extent arbitrary. The following four divisions of the series are commonly referred to:

1. Common or fat limes
2. Hydraulic limes

3. Hydraulic or Roman cements
4. Portland cement.

The points of distinction between 1 and 2 have been noted. In composition, 2 and 3 are not separable. By an increased temperature in burning some hydraulic limes will become hydraulic cements. Twenty-four per cent of clay is about the permissible upper limit for the hydraulic limes, while Roman cements are in use which contain but little over twenty per cent of clayey impurities. The chief distinguishing feature of these two groups lies in the ability of the limes to slake to a paste with water without previous pulverization. Fine grinding is necessary before water will affect appreciably Roman cements and before they will harden as a mortar.

contain but little over twenty per cent of clayey impurities.

The feeble hydraulicity of the limes and the relatively strong of the cements would appear to be due to the varying degree to which chemical combination has been brought about in burning. In lime burning, little if any chemical action occurs between the lime and the clay. What does take place tends to produce an unstable or 'unlocked' condition of the clay and other siliceous materials such, that in the presence of water, the lime hydrate slowly attacks these and combines with them to form silicates that are harder and more durable than lime mortar. Clinkering in burning, is an indication of chemical action, further progress in rendering available and susceptible to the attack of the lime and water the clay and other siliceous substances in the stone. Burned to this condition, the product is properly termed a cement and in use attains a stony hardness and relatively great permanency.

Hydraulic or Roman cements are spoken of also as natural or rock cements, since they are made from limestones in which the ingredients occur naturally in the proper proportions. Such limestones are found in different parts of the United States, but have been utilized principally in the Appalachian states of the East and along the Ohio river. The actual composition ranges between wide limits as shown in the table below in which are compiled the analyses of five reputable brands of natural cements.

ANALYSIS OF NATURAL CEMFNTS.

NAME	Silica	Alumina	Ferric Oxide	Lime	Magnesia	Alkalis	Loss on Ignition
"Fern Leaf" brand Louisville, Ky.	26.40	6.28	1.00	45.22	9.00	4.24	7.86
N. L. & C. Co. Rosendale, N. Y.	30.50	6.84	2.42	34 38	18.00	3.98	3.78
"Hoffman" Rosendale, N. Y.	27.30	7.14	1.80	35.98	18.00	6.80	2.98
"Utica" Brand Utica, Ill.	27.60	10.60	0.80	33.04	7.26	7.42	2.00
Mankato, Minn.	28.43	6.71	1.94	36.31	23.89	1.80	0.92

Numbers 3 and 4 in the outline on page 6 bear to some extent a similar relation to that briefly given for 2 and 3. A more complete vitrification of the ingredients in the cement mixture until they issue from the kiln as thoroughly vitrified clinker produces the maximum amount of hydraulic silicate. The chemical changes which occur in burning are complicated and become more so the higher the temperature over that employed in the manufacture of natural cement. Just what these changes are is not accurately known, but experimentation has determined within fairly narrow limits what proportions of the various constituents entering into a mixture of clay, silica and limestone will produce the greatest amount of unstable, hydraulic silicate, and what temperatures are required to accomplish this result. These proportions and temperatures are employed in the manufacture of Portland cement.

The foregoing remarks will serve to show the relation of limes as a mortar material to other substances used for a similar purpose. This paper has to do with limes alone and the several physical properties of the latter that are of chief importance will be briefly discussed.

SLAKING.

The property belonging to limes which makes them of industrial value is their ability to slake or crumble to a powder on the addition of water, and to harden when allowed to stand in contact with the atmosphere. The reaction which occurs in slaking has already been given. If a lime is properly burned, all lime carbonate in the original stone has lost its carbon dioxide, and become quicklime (CaO). It is the rapid change accompanied by the evolution of heat when water is added that causes lime to slake.

Slaking is a physical evidence of the hydration of lime, but it is not to be understood that slaking is a necessary result of such chemical action. The two processes are really distinct. The exposure of caustic lime to a moist atmosphere occasions slow hydration, accompanied by crumbling to a powder. Along with this change occurs an increase in volume of about one and three-quarters times that of the original lime. Such lime is air-slaked and is largely changed to the hydroxide $Ca(OH)_2$. If this lime be exposed to water, it will further increase in volume, but the paste resulting will be sharp and sandy in texture, and of much less value for mortar purposes than freshly slaked lime. In this case a portion of the CaO has no doubt combined with the CO_2 of the air, so that air-slaked lime is actually a mixture of lime hydrate and carbonate. It is possible also to bring about the complete hydration of lime by steam at temperatures above boiling, without any change of volume or any sign of crumbling.

Slaking may, therefore, be defined as the hydration of calcium oxide, quicklime, accompanied by an increase in temperature and volume. The increase in temperature is caused by the combination of the lime and water. It is an exothermic reaction, one in which heat is evolved. Whether or not this heat becomes evident depends on the vigor and rapidity of the reaction.

Slaking is commonly accomplished by the addition of sufficient water to cover the lime, and by further additions as needed. It is desirable from the practical standpoint that the greatest possible increase in volume be secured in slaking. This is accomplished by careful control of the amount of water throughout the process. The evolution of heat in such quantities as to generate steam within the mass is a necessity to the slaking process. At the same time, more water than simply that required for hydration is essential. It is the expansion of the steam between the molecules of hydrating lime which forces them apart and caused the mass to crumble. As the particles are separated, the surrounding excess of water acts to remove them, as in the case if any fine sediment, and as they float away in partial suspension, new surfaces of the lime are constantly exposed. A large excess of water prevents proper slaking by keeping the temperature so low that the necessary steam does not form. The mass then expands poorly, slakes slowly, and the product is lumpy. The lime is said to be 'drowned.'

The result of too little water is a 'burnt' lime. In this case, the water forms a gelatinous film of hydroxide over the surface of the lumps which dries down enclosing caustic quicklime in the center, and so clogs the pores that further progress is much retarded or prevented. When too litle water is used, the action is apt also to be so violent in the case of 'fat' limes, that much or all of the moisture passes off as a vapor, because of the excessive temperature developed. This frequently leaves the lime but partially hydrated, dry, and imperfectly slaked.

Dolomitic limes slake more slowly and much cooler than do high calcium limes. The heat generated is due to the hydration of the calcium oxide, the magnesia remaining as the oxide during slaking. Although magnesium carbonate loses CO_2 at a lower temperature in burning than does the carbonate of lime, it hydrates only with difficulty and probably passes directly from oxide to carbonate in the hardening process. It is thus necessary to add the water required very gradually in slaking dolomitic limes in order to avoid 'drowning' and to secure the best results.

The proper amount of water to use varies and can be ascertained for each individual lime only by actual trial. It is usually found more satisfactory to add the water in several different portions as slaking progresses, especially with the lean, slow slaking and dolomitic limes. In this way, by a little attention, the temperature of the slake can be controlled so that the best product is obtained from the lime in use.

The expansion of volume in slaking may be as high as three and one-half times with pure white limes. It is found to range from two and one-half to the figure named. Lean, so-called hydraulic limes, and dolomitic limes expand less. Increase in volume is ordinarily estimated by a comparison of the bulk of the dry quick-lime and of the paste after slaking. Careful experiments with samples of both high calcium and dolomitic limes made by the Ohio Geological Survey* show an increase in apparent volume for the white limes of from one hundred thirty-six per cent using twenty per cent less water than theoretically necessary for hydration, up to two hundred sixty-four per cent with forty per cent excess of water. With three hundred per cent excess, the increase was but forty-five per cent. The comparison was made between the apparent volumes of the ground quick-lime and of the dry hydrate produced. Under the same conditions, a dolomitic lime gave one hundred ninety-three per cent expansion with a deficiency of twenty per cent of water, of two hundred ten per cent with the exact theoretical quantity of water, and of but about twenty per cent with an excess of water. The increase in volume is decidedly in favor of the white lime, the smaller expansion of the dolomitic lime being accounted for no doubt, by the fact that the magnesia takes up very little water in the slaking process.

The actual increase is, as a matter of fact, more apparent than real. The calcium hydroxide produced from a weighed amount and accurately determined volume of calcium exide will occupy a space but thirty-five to forty per cent greater than the volume of the oxide. Few experiments have been made along this line and the above figures were obtained with a high grade white lime by the use of the Seger volumeter.

If allowed to stand in the air lime deteriorates by the process of air-slaking already described. It also slowly absorbs carbon dioxide, which renders it of little value for mortar. After slaking, if the paste is not to be used at once, it should be protected from the atmosphere, since moist lime hydrate changes very readily to the carbonate by the absorption of CO_2. Slaked lime is very commonly buried so as to be covered with several inches of soil where it will keep for months without deterioration.

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* S. V. Peppel, Bulletin 4, Ohio Geol. Sur. (4th Series), p. 337.

Owing to the susceptibility to deterioration of the high calcium limes on the one hand, and the exceeding slowness with which dolomitic limes slake on the other, so called 'hydrated limes' are being put upon the market. The quick-lime is subjected to a partial hydration or slaking at once after burning and before being sacked or barreled. The completeness of the hydration in the case of five Ohio* products ranged from fifty-eight to ninety-four per cent, one hundred being taken as the best that is possible on a commercial scale. Specially designed and patented apparatus and processes are being employed in the hydration of limes, but it is believed that such special equipment is not necessary nor will the preparation of hydrated limes, without doing so under a patent, make any person liable for infringement. At present but a single plant produces hydrated lime in Iowa. Others are contemplated in the near future.

The desirable qualities of hydrated lime are (1) its convenience in use, for it is already pulverized and but little time is required to make a mortar by mixing the ingredients dry before adding the water; (2) its lasting qualities, as it will keep much longer without detriment than the unslaked product. Magnesian limes are more commonly prepared in this way, and the saving of time in their use is a very important commercial consideration. Hydrated magnesian lines are found by the Ohio Survey** to have specific gravities of 2.12 to 2.25. High calcium limes run about 2.45. A series of tests with an Iowa white lime gives specific gravities from 2.2 to 2.32 for the slaked lime, while the quick lime is 2.08.

SETTING AND HARDENING.

In slaking, the lime takes water into chemical combination, and becomes the gelatinous hydroxide. When this hydroxide is put in place as a mortar, it is said to set. This preliminary set is due to the loss of the water used in mixing which brings about a certain rigidity in the same way so far as is known, that clay becomes hard on drying. Part of the moisture evaporates from exposed surfaces, but the larger proportion is in most instances absorbed by the porous brick or other masonry material with which it is used. The more rapid the set, that is, the more rapidly the mortar loses its water, the safer the construction, providing the proportions of sand and lime are such that shrinkage may be left out of account.

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* S. V. Peppel, Bulletin 4, Ohio Geol. Sur., pp. 335-336.

**S. V. Peppel, loc. cit.

A second process begins at once when the lime is exposed to the air. This is the absorption by chemical combination of carbon dioxide by which the lime returns to the carbonate condition, as it existed in the original limestone. The process is a slow one and may require years for completion, but this depends largely on the surface that is exposed and the thickness of the layer and porosity of the mortar. A large number of chemical tests on small briquets having a minimum cross section of one square inch, made with mixtures of sand as high as 6 to 1, and allowed to stand for a maximum period of one year, showed none in which carbonation was complete. This action is most rapid in the first few months until a crust of the carbonate forms on the exterior. The crust retards the process and at the same time protects the soluble hydrate within from being dissolved. The interior portions of large masses may therefore, never reach this final condition in hardening.

Long contact of lime hydrate with finely divided silica is known to cause a reaction by which the silica combines with the lime forming a stable silicate of lime. The extent to which this reaction progresses depends on the physical and chemical qualities of the siliceous impurities in the lime or of the sand used with it. If these are very fine, chemical action is favored. Silicates, such as clay of feldspar, for example, are more susceptible to attack by the lime than is quartz sand. Hydraulic limes are apt therefore, other things being equal, to give a more durable final product than the purer limes. In the same way, muddy or clayey sand used with lime, although less desirable at the start, will likely contribute to the durability of the mixture in time because of the development of these stable compounds by the caustic action of the lime. In the case of silicates, it is probable that other elements, especially alumina, also enter into combination.

LIME MORTAR.

Sand.—Lime has a variety of uses in various industries but by far its most important application is and has been as a mortar in structural work, interior wall plastering, etc. For these purposes, slaked lime alone can not be used on account of the great shrinkage of the lime paste in setting and its lack of inherent strength when set. It is, at the same time economical to add some foreign material which is cheaper than the lime itself. The filling material commonly employed is sand. Most sands are composed largely of quartz grains, although fragments of feldspar and of many other minerals are often found in varying amounts with quartz. There is often also more or less of earthy or clayey matter in sands.

In general, it may be said that the composition of the sand is not an important consideration. Any inert substance which does not shrink nor deteriorate may be used. Ground limestone, for example, or the pulverized sand from any durable rock will serve the purpose equally well.

The physical condition of the sand is, however, of considerable importance. The function of the lime is to serve as a binder to hold the particles of the aggregate together. If these particles are angular and rough, they afford good surfaces for the attachment of the lime. Sharp sand will therefore make a stronger mortar than one composed of rounded grains. Only sufficient lime is required to fill the voids and to form a thin film around each grain of the aggregate. The more nearly the voids are filled with the sand grains themselves, in other words, the smaller the percentage of pore space in the sand, the less the amount of lime needed. A sand composed of a properly proportioned range of sizes of grain will therefore not only give the strongest product but will do so with the least amount of lime. Few sands as they occur naturally are composed of the proper range of sizes to give the smallest pore space. It is sometimes not difficult, and often may be a matter of economy, to correct a poorly proportioned sand by screening or by the addition of suitably graded materials. The voids in a sand are determined readily with simple apparatus.* Separation into a series of sizes is quickly done by sieves of a number of different meshes. These two tests afford data as to how far a given sand departs from the ideal mixture of grains and indicate the size of grain and quantity to be added for correction.

The sand grains should be practically in contact throughout the mass so that the lime paste forms merely a plastic film filling the interstices. Such a mortar when it has attained its final hardness may properly be regarded as sandstone in which the cementing matter is lime carbonate. It differs from the natural stone only in its position and origin, being as strong, if properly made, and as durable as that quarried from natural ledges.

White limes shrink much more in drying than do dolomitic limes. For this reason is it more highly important that the proper proporton of sand be used with the former. The binding or adhesive power of white limes is also less. This is evidenced in walls where the mortar separates readily from the brick and can itself often be crumbled in the fingers. Such defects are believed to be due more to poor mixing and wrong proportions of sand and lime than to any inherent quality of the lime itself. On the other hand, dolomitic limes possess great adhesive strength and not only form a denser mortar by binding the sand particles firmly together, but contribute towards the stability of the wall by adhering to the brick or stone.

TESTS OF LIME MORTARS.

Although lime has been used as a mortar since very early times, and is of late being employed in various other ways in structural engineering work, few records of tests of those physical properties which make it of value are to be found. The purpose of the following series of tests is to investigate several of the physical properties of lime mortars, including the following points: (a) The influence of slaking with increasing amounts of water; (b) the increase in strength with increased setting periods; and, since in practice, limes are seldom used in the neat condition; (c) the effect of varying proportions of sand on the strength of the mortar, and the rapidity of setting. There has also of late been considerable discussion as to the relative merits of the white or high-calcium limes, and the brown or magnesian limes.

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* Standard Sand for Cement Work, M. J. Reinhart, Proc. 3d Ann. Convention Iowa Association Cement Users, Iowa Engineer, Vol. VII, No. 1, p. 34.

To obtain definite data on these several points, the following plan was adopted in the beginning: Barrel samples of commercial limes were obtained from the principal producers in Iowa, and a few from bordering states. Samples of white lime were tested from Springfield, Mo., and Mason City, Iowa; of dolomitic lime from Viola, Iowa; Mason City, Iowa; Maquoketa, Iowa; and Eagle Point, Iowa.

While it was evident that the factors enumerated above were the important ones to be studied, with each lime it was necessary to carry on considerable preliminary experimenting in order to be able to lay out an exact systematic method of procedure. A provisional line of experiments was therefore initiated, using the white lime from Mason City. The quick-lime was slaked with percentages of water ranging from the amount which would produce a dry powder as a minimum, to a maximum of 300 per cent by weight. Slakings were then made with 100, 150, 200, 250, and 300 per cent of water, calculated on the basis of dry quicklime as 100 per cent. From each slaking, series of briquettes were made with the following sand dilutions,* by weight:

One part sand to 1 of lime, 1½ of sand to 1 of lime, 2 of sand to 1 of lime, and so on to 5 parts of sand to 1 of lime. Four sets of briquettes were made from each sand mixture to be broken at the end of four, eight, twelve and sixteen weeks respectively. Ten briquettes were used in each set from which to obtain an average.

Briefly then, from the lime paste obtained by slaking in each of six different percentages of water, briquettes were made with nine different dilutions of sand. Since four periods of set were to be allowed, with ten briquettes to be broken at the expiration of each period, a little arithmetical calculation will show that for each lime tested, according to this plan, 2160 briquettes would be made. As the work progressed, it was soon discovered, however, that this general scheme required more or less modification according to the peculiarities of each particular lime.

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*A standard river sand was used throughout the tests whose grains passed twenty mesh (linear) and remained in a thirty mesh sieve.

HIGH CALCIUM WHITE LIMES.

Lime from Mason City, Iowa.

The limestone from which the Mason City lime is produced comes from the Devonian beds, and has the following composition:

	Per cent
Water	0.51
Insoluble	0.63
Alumina and iron oxide	0.71
Lime (CaO)	54.59
Carbon dioxide	42.89
Magnesia (MgO)	0.47
Carbon dioxide	0.52
	100.32

Analysis of the commercial product:

	Quick-lime.	After slaking.
Insoluble	1.02	0.60
Alumina and iron oxide	2.98	2.80
Lime (CaO)	95.40	71.10
Magnesia (MgO)	0.43	0.40
Loss on ignition	0.00	25.60
Totals	99.83	100.50

A sufficient quantity of quick-lime was slaked at one time to make the full number (360) of briquettes as planned for each percentage of water. Precaution was taken in slaking to add the water in such quantities and to agitate the mass so as to facilitate the process and to obtain the greatest increase in volume with the amount of water employed. The lime paste was allowed to stand for twenty-four hours until all signs of heat had disappeared and then put into air tight jars to be used as needed. All weights were calculated on a dry basis, the moisture being determined before each batch of both sand and lime was weighed out. In the case of the higher percentages of water, it was necessary to drive off by careful heating, care being taken to keep the temperature below boiling, some of the excess water, in order to reduce the paste to a workable consistency. One man did the work, using his judgment to obtain as nearly as possible the same consistency in every batch. Forty briquettes were made from each mixing. The briquettes remained in the molds until they could be safely removed, after which they were placed on edge on open shelves and allowed to harden for their respective periods.

The tensile strength test was adopted as a means of obtaining comparable results more because of its convenience and the uniform treatment to which each lime would be subjected, than as representing conditions which lime mortars would meet in actual use. As noted earlier, the principal function of a mortar is to serve as a matrix or adhesive to bind together particles of aggregates and sections of masonry structure. Adhesion, therefore, and crushing strength tests would give more direct results. Such tests have not as yet been made.

Records of tensile strength tests of lime mortars are to be found in the Report of the Secretary of War for 1896, Document No. 2, Volume II., part 5, on page 2839. These tests were made with paste slaked with three hundred per cent of water, ratios of sand from 3:1 to 17.7:1, and setting periods of twenty-eight and twenty-nine days and three months. Average strengths range in the short time tests from sixty-four pounds with a ratio of 8.8:1 to twenty pounds per square inch with a sand lime ratio of 17.7:1 and, in the three months tests from seventy-one pounds with a ratio of 8.8:1 to thirty-six pounds with a ratio of 17.7:1. Tensile strength has also been investigated by the Ohio Geological Survey.* The following table, page 18, will indicate the results obtained.

The period of set allowed in these tests is entirely too short for valuable results. But one briquette seems to have been made for each percentage of water. It is evident that a much larger number should be used to obtain an average figure.

A similar line of experiments made by Mr. George S. Mills, of Toledo, Ohio,† affords results which bring out quite clearly the relative strength of the white and brown

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*S. V. Peppel, Bulletin 4, p. 337.

†Municipal Engineering, Vol. 28, p. 6.

PHYSICAL TESTS OF OHIO LIMES.

KIND OF LIME	Tensile strength of mortar after 7 days. Mortar made by adding 4 volumes of sand to 1 volume of quick-lime.	REMARKS
High calcium or white lime	48.95	Water 20% less than theory for complete hydration.
	70.6	Theoretical amount of water
	50.	100% excess. Product, moist powder.
	42.36*	*Broke badly. Defective briquette. 100% excess. Heat applied in slaking.
	48.95	200% excess, moist, lumpy mass.
	65.90*	*Briquettes cracked before before going into machine. 300% excess. Smooth stiff putty.
Dolomitic or brown lime	24.48*	*Bad briquette. 20% less water than theory for complete hydration.
	72.2	Theoretical amount of water
	58.*	*Bad briquette. 20% excess
	81.90	40% excess
	82.84	Sticky, lumpy mass. 100% excess
	68.*	*Bad briquette. Stiff putty. 200% excess

limes and the relation of strength to progress in setting and hardening of the mortar. The mortar was made with two parts sand to one of slaked lime by weight. The strength is expressed in pounds per square inch. From four to six breaking strengths were used for each time period to obtain the average results given in the table.

	1 mo.	2 mo.	3 mo.	4 mo.	6 mo.	1 Year
Dolmite Lime		28.8	37.2	51.	83.	92.8
High-calcium Lime	30.7	36.6	39.2	39.	50.8	44.6

The Mason City lime is a hot lime, which slakes vigorously and requires constant attention when water is given to it. The quantity of water which would just leave the hydrate practically a dry white powder when it had cooled to the atmospheric temperature was found to be about 75 per cent of the weight of the dry hydrate.

Table 1 below gives in detail the breaking strength of the briquettes made with the Mason City lime. A Fairbanks standard testing machine was used.

From these figures, three sets of curves can be constructed which will bring out in a graphic way the general trend of the results. One set, with sand-lime ratio and average tensile strengths as coordinates; a second set using the setting periods and tensile strength; and a third, based on the percentage of water used in slaking and the tensile strengths.

Inspection of the table and of the accompanying curves will show that, in general, the strength increases with increase of sand up to 2:1, 2.5:1, and possibly 3:1 in some cases and that beyond these ratios it decreases. Considered with reference to the effect of different periods of set on the strength, little more is to be observed than an increase in all instances during the first two months. With the longer periods the change is not sufficiently pronounced to afford a characteristic curve.

It is evident that the lengths of time allowed for the lime to set were too short for the maximum strength to be attained. This conclusion is supported by chemical determinations of the carbon dioxide absorbed which show that carbonation is in no instance even approaching completion at the expiration of sixteen weeks. Sufficient time should be allowed for the longest period for all the lime to return to the carbonate condition. Then the curves constructed according to the first two methods mentioned above would indicate clearly the progress of the gain in strength according to percentage of sand and setting period.

Curve sheet No. 3 shows the relation between the percentage of water used in slaking and the tensile strength of the briquettes. They are arranged in groups of four curves, each according to the whole-numbered sand-lime ratios. The general aspect of the five groups taken in conjunction with the tables, indicates a rapid rise in strength at one hundred to one hundred fifty per cent of

TABLE 1 Slaked in 75% of Water

Time of set in weeks	Ratio Sand to Lime	Lbs. per sq. in.			Ratio Sand to Lime	Lbs. per sq. in.			Ratio Sand to Lime	Lbs. per sq. in.		
		Average	Maximum	Minimum		Average	Maximum	Minimum		Average	Maximum	Minimum
4	1:1	48.4	51.4	38.5	1½:1	45.7	50.5	39.5	2:1	54.5	64.4	46.1
8		50.6	56.3	47.5		40.7	44.5	35.0		54.3	63.3	48.0
12		56.1	60.4	45.0		47.1	50.0	44.3		48.5	52.5	46.0
16		53.9	50.0	42.0		46.2	49.4	40.8		52.0	55.9	41.2
4	2½:1	55.1	60.4	50.4	3:1	50.6	56.2	41.9	3½:1	48.7	55.0	30.0
8		54.6	58.6	51.0		52.6	58.4	44.5		52.1	56.0	46.5
12		46.7	48.5	43.5		48.1	51.0	45.9		48.0	54.3	42.7
16		52.3	56.9	49.5		43.4	48.6	33.6		32.9	53.0	21.5
4	4:1	47.2	51.4	43.5	4½:1	43.7	53.0	35.0	5:1	43.6	58.4	38.0
8		47.8	57.0	37.0		41.8	52.5	37.0		45.9	50.0	34.3
12		43.6	49.5	33.7		42.6	46.6	38.4		42.5	51.4	28.6
16		41.5	49.0	28.8		44.6	50.0	33.0		53.1	55.7	50.0
4	5½:1	37.1	44.1	28.8	6:1	36.0	39 6	34.6				
8		37.8	45.0	32.6		35.9	39.0	31.0				
12		38.5	44.1	33.3		33.1	39.4	29.8				
16		38.6	40.8	34.9		35.4	41.6	30.0				
Slaked in 100% of Water												
4	1:1	47.3	56.2	39.7	1½:1	57.1	61.9	53.0	2:1	55.9	73.6	45.4
8		56.4	64.8	48.9		60.5	68.7	48.5		72.3	94.7	55.1
12		64.9	76.7	52.5		64.4	85.8	45.4		76.2	89.8	54 1
16		62.9	73.3	53.6		73.2	91.6	55.6		67.9	83.6	53.1
4	2½:1	54.9	62.5	43.8	3:1	49.9	65.6	40.9	3½:1	51.0	61.3	44.5
8		64.6	80.4	53.5		57.0	67.6	51.5		54.0	62.8	49.4
12		55.0	67.7	41.0		54.6	63.6	45.0		55.5	68.7	50.0
16		57.6	74.4	36.7		53.1	58.1	40.6		56.8	65.7	51.5
4	4:1	52.8	59.7	41.2	4½:1	47.6	55.4	38.8	5:1	64.1	75.4	52.5
8		53.8	57.0	50.0		49.6	53.0	44.5		68.1	74.5	65.0
12		52.2	58.7	45.4		54.4	58.0	48.5		63.7	69 3	57.0
16		49.6	58.6	43.0		48.7	52.0	43.0		65.8	72.5	59.0
4	5½:1	56.4	64.7	52.0	6:1	53.7	59.2	49.5				
8		56.7	62.0	51.5		52.5	54.5	50.5				
12		60.4	73.0	53.4		54.7	60.0	49.0				
16		50.4	59.4	41.0		47.8	55.0	31.3				
Slaked in 150% of Water												
4	1:1	41.0	48.3	33.8	1½:1	47.1	49.9	36.1	2:1	45.0	53.2	38.0
8		40.9	48.9	37.0		47.6	64.6	36.1		46.9	56.7	40.4
12		45.8	55.3	34.0		50.8	59.8	40.4		52.1	68.3	39.0
16		47.6	55.7	39.2		57.5	74.5	40.8		62.0	81.6	51.5
4	2½:1	55.4	68.7	38.8	3:1	60.7	68.7	45.7	3½:1	71.9	81.0	61.8
8		59.8	78.1	44.4		54.0	64.5	47.5		60.7	71.4	41.4
12		56.6	76.3	42.8		63.7	72.1	51.0		76.2	84.1	69.7
16		55.4	79.9	51.0		77.3	83.0	66.3		76.3	83.4	64.3
4	4:1	60.5	67.7	53.5	4½:1	51.7	59.4	44.9	5:1	46.0	51.5	40.4
8		63.9	74.5	59.4		58.9	62.6	53.1		47.5	51.0	45.0
12		69.9	76.8	60.6		61.1	64.3	49.2		49.6	48.2	42.0
16		59.5	70.7	49.5		60.5	68.4	53.1		60.2	75.5	42.3

(Continued next page)

Slaked in 200% of Water

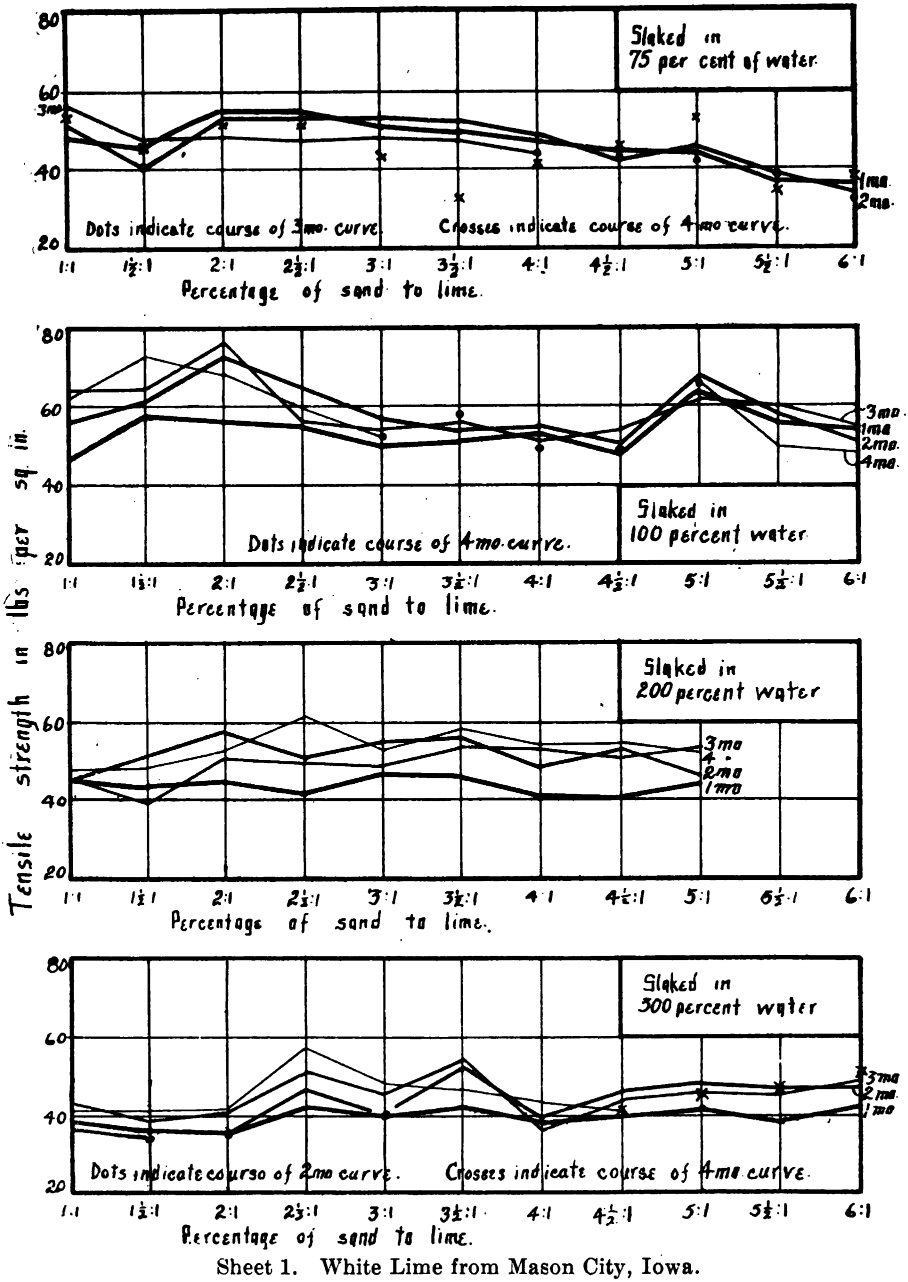
Time of set in weeks	Ratio Sand to Lime	Lbs. per sq. in.			Ratio Sand to Lime	Lbs. per sq. in.			Ratio Sand to Lime	Lbs. per sq. in.		
		Average	Maximum	Mini mum		Average	Maximum	Mini mum		Average	Maximum	Mini mum
4	1:1	45.0	60.6	37.2	1½:1	44.4	58.3	35.7	2:1	45.2	54.6	38.5
8		45.5	62.8	44.3		51.0	65.6	43.9		57.3	69.7	45.9
12		45.2	61.8	42.6		40.0	57.1	34.7		50.6	61.2	40.0
16		47.4	53.1	41.2		48.5	67.0	36.4		53.4	62.8	46.5
4	2½:1	41.7	55.2	33.4	3:1	46.9	57.1	37.0	3½:1	46.2	53.1	38.0
8		51.2	69.8	38.8		55.0	72.4	43.9		56.0	67.0	46.5
12		50.9	61.8	37.4		49.6	65.9	42.3		55.2	59.2	52.0
16		60.6	68.0	53.1		54.0	58.2	48.4		57.8	60.8	52.1
4	4:1	40.6	44.9	38.0	4½:1	41.0	44.1	38.6	5:1	43.8	46.0	39.8
8		47.7	57.3	38.8		53.2	63.1	38.8		46.3	50.5	42.6
12		52.9	60.0	47.5		52.2	57.4	45.5		53.5	59.6	38.0
16		53.0	57.0	45.5		54.9	57.1	53.0		52.7	54.5	47.0

Slaked in 250% of Water

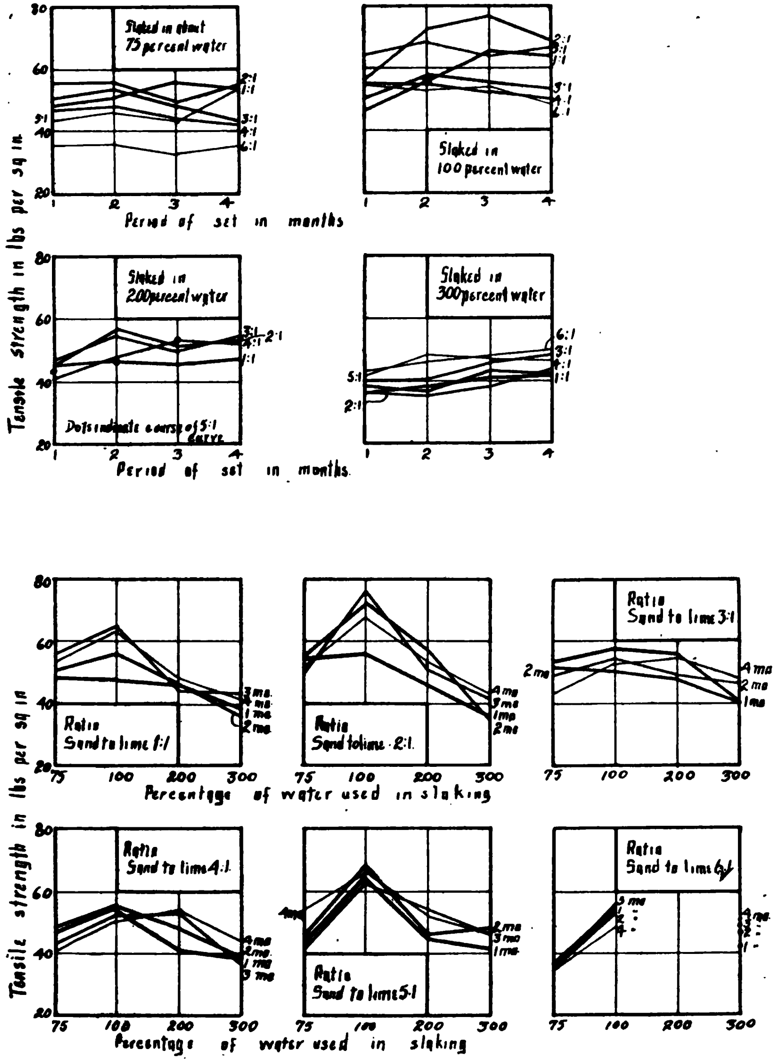
4	1:1	35.4	37.6	33.2	1½:1	38.9	40.9	33.0	2:1	37.9	41.0	33.0
8		43.2	49.0	36.0		45.4	53.2	35.4		48.6	56.1	44.1
12		38.0	43.9	35.0		41.2	46.4	36.8		47.4	55.5	42.8
16		40.0	45.2	36.0		43.9	50.0	36.5		51.8	58.9	42.0
4	2½:1	42.7	62.5	38.8	3:1	52.6	61.0	42.9	3½:1	52.7	60.0	43.4
8		50.1	60.2	36.5		54.8	63.6	41.0		48.5	55.1	40.4
12		51.9	63.9	40.4		56.8	70.6	45.2		56.3	63.3	46.0
16		58.9	70.8	46.4		58.7	69.0	50.0		58.0	62.0	57.0
4	4:1	43.2	64.0	35.0	4½:1	47.5	58.3	37.4	5:1	50.2	55.4	30.5
8		43.4	48.0	35.0		50.9	58.0	35.9		56.6	62.2	42.4
12		53.0	56.6	49.0		54.5	60.0	45.5		56.9	61.6	53.0

Slaked in 300% of Water

4	1:1	38.1	47.4	25.5	1½:1	35.6	48.4	26.0	2:1	36.0	48.4	28.6
8		35.8	54.2	27.3		35.2	53.6	28.8		35.9	43.9	28.6
12		43.0	58.3	28.2		38.5	54.6	27.8		40.7	53.5	34.0
16		41.7	57.8	26.6		40.8	51.0	34.3		41.2	52.1	32.0
4	2½:1	41.8	50.0	35.3	3:1	40.0	50.5	32.6	3½:1	42.4	50.0	33.3
8		46.2	51.0	40.8		40.2	53.0	30.6		52.1	58.1	45.2
12		51.1	60.2	39.6		46.1	51.0	36.0		52.5	55.0	45.9
16		56.8	63.8	50.0		47.8	54.0	40.0		46.7	58.0	35.0
4	4:1	38.2	46.5	31.3	4½:1	39.7	43.5	22.8	5:1	40.7	47.5	35.0
8		39.2	51.5	31.9		45.8	55.5	38.2		48.3	54.9	42.7
12		37.6	43.5	28.8		43.6	48.5	35.9		46.4	52.0	43.0
16		43.0	50.5	40.0		41.4	49.0	30.7		46.1	53.0	30.0
4	5½:1	37.8	44.5	34.0	6:1	41.7	46.0	36.6				
8		46.7	52.9	38.6		46.5	51.3	41.5				
12		46.5	52.4	38.5		47.7	54.3	43.1				
16		46.5	58.0	43.2		49.8	55.0	44.3				



Sheet 1. White Lime from Mason City, Iowa.



Sheets 2 and 3. White Lime from Mason City, Iowa.

water, and as rapid a fall at two hundred which continues through the higher percentages. It is to be noted that one hundred per cent of water gives the highest strength with the 1:1, 2:1 and 5:1 sand dilutions, while with the two middle members of the series, 3:1 and 4:1, the highest strength is reached with one hundred per cent. This may be but a coincidence, with results for only one lime at hand. The percentage theoretically necessary to hydrate the calcium oxide in the lime is but about thirty per cent of the weight of the dry quick-lime. The increase in strength with successively longer periods of time can also be traced from these curves. It will be further noted that the rapidity of increase is greater the higher the percentage of sand used.

The experience gained and the results accruing from the foregoing tests of the Mason City white lime suggested certain changes in the general plan of the experiments. In all the tests whose results follow, the longest setting period is made one year, and sets of briquettes were broken at the end of three, six, nine and twelve months. The percentages of water used for slaking are the even hundreds up to three hundred per cent as a maximum, with the exception of the white lime from Springfield, Mo., in which case four hundred per cent of water was used, since it slaked to a dry powder with one hundred per cent. The lowest percentage is in all instances the largest possible amount that would leave the slaked product a dry powder. Only the whole-numbered sand-lime ratios are used in the later tests.

WHITE LIME FROM SPRINGFIELD, MISSOURI.

The limestone from which the Springfield lime is made has the following chemical analysis.*

	Per cent
Lime Carbonate ($CaCo_3$)	**99.46**
Iron Oxide (Fe_2O_3)	**0.21**
Silica (SiO_2)	**0.33**
	100.00

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*20th Annual Rep. U. S. Geological Survey, Part VI (Continued) p. 415, also Bulletin 44, N. Y. State Museum, p. 924.

Analysis of the commercial product:

	Quick-lime.	After slaking
Insoluble	1.00	0.67
Iron and Alumina (Fe_2O_3+Al_2O_3)	1.80	1.11
Lime (CaO)	94.70	73.20
Magnesia (MgO)	0.40	0.43
Loss by ignition	2.08	24.25
Carbon Dioxide (CO_2)	Trace	.....
	99.98	99.76

The Springfield lime is unusually pure and, as may be expected, slakes rapidly and with much heat. With two hundred per cent of water, it was fonud difficult to prevent the lime from 'burning.' Up to three hundred per cent of water, it was impossible to keep the paste thoroughly mixed on account of the generation of steam and the violence of the slaking action. Three hundred fifty to four hundred per cent work best and give a uniform, well slaked product.

It is unfortunate in any investigation if the person beginning the work can not carry it to completion. Even though the training of the experimenters be identical, personal equation always enters, and sometimes to a sufficient extent in small matters of manipulation, and in the exercise of judgment, to produce unexplainable irregularities in results. In the present experiments it was necessary to place the work in new hands at times during their progress. Lack of uniformity in the data which follow can in some instances be accounted for only by such changes, and yet it is not possible to assign absolutely to this cause variations that appear. Notwithstanding all minor deviations from the rule, however, there are revealed certain general truths that are brought out in the results given.

In Table II are compiled the results of the tests of the Springfield lime and these are graphically shown in sheets 4, 5 and 6 which follow.

The curves on sheet 4 show that there is little change in strength after three months. In a few instances there is a slight gain, but in most cases the three, six and nine months figures run close, followed, as a rule, by a falling off at the end of twelve months. Inspection will show that this falling off is most pronounced with the high percentages of sand, 4:1, 5:1, 6:1, where the decrease has frequently begun at the end of three months.

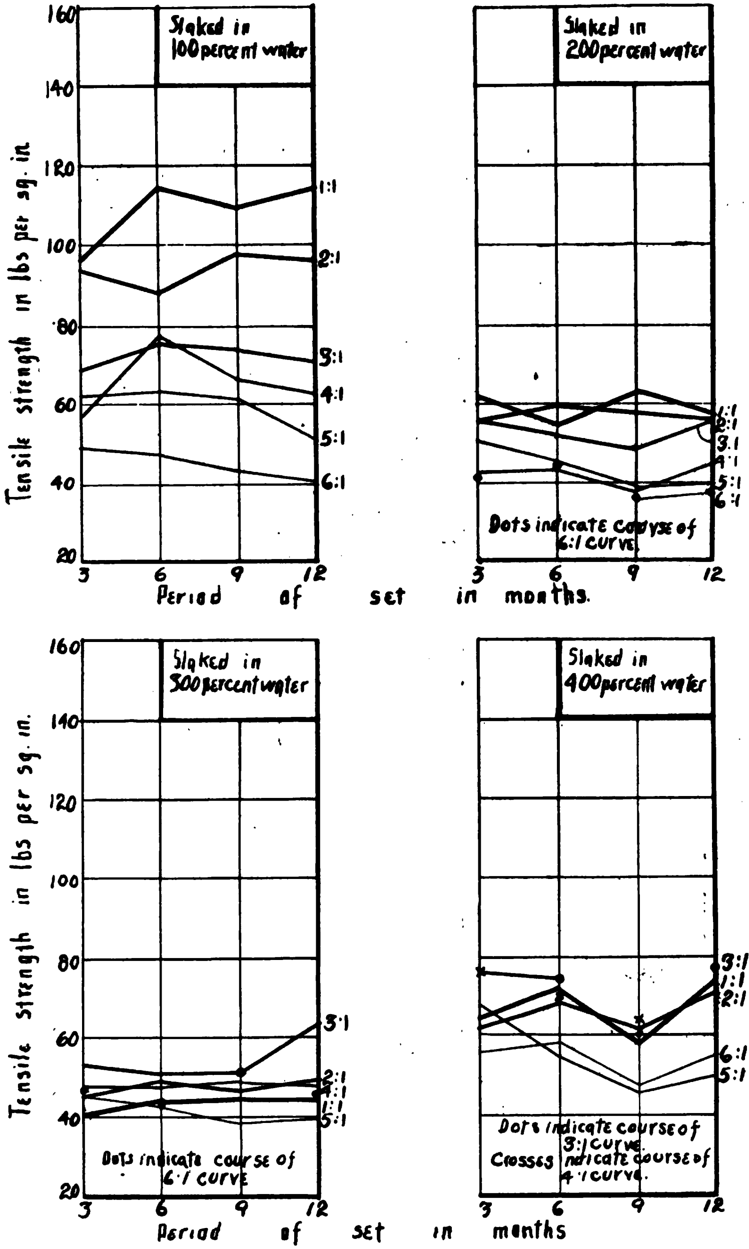
The diagrams according to percentage of water (Sheet

TABLE II

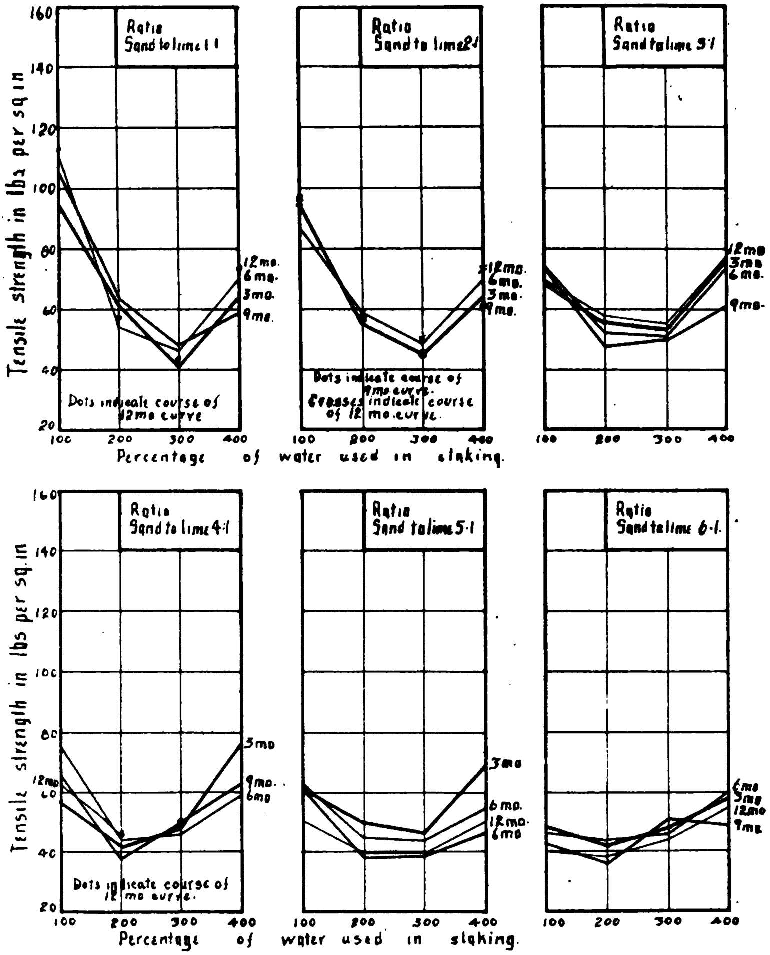
SPRINGFIELD, Mo. WHITE LIME

Slaked in 100% Water

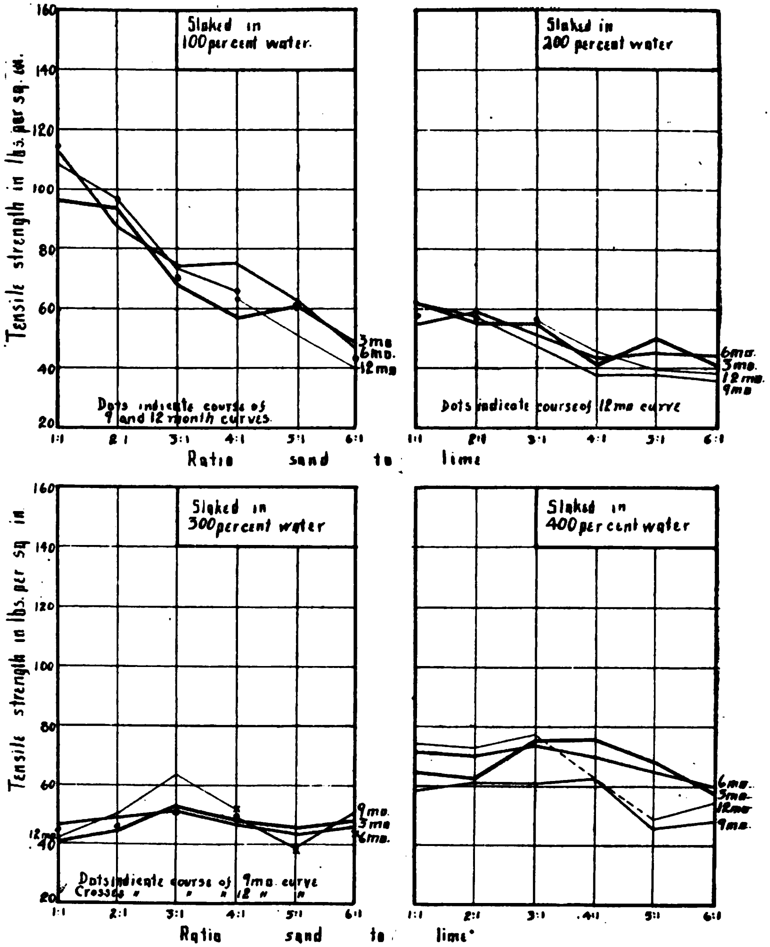
Time of Set in Months	Tensile Strength in Pounds Per Square Inch																	
	Sand to Lime 1:1			Sand to Lime 2:1			Sand to Lime 3:1			Sand to Lime 4:1			Sand to Lime 5:1			Sand to Lime 6:1		
	Av.		Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.
3	96.2	112.	67.	93.95	107.7	81.7	68.24	77.6	49.5	56.71	60.	50.	61.4	65.	58.8	48.21	55.	40.
6	113.7	121.	102.	88.23	105.5	72.1	75.4	96.1	61.5	75.60	82.1	69.2	63.06	68.9	53.4	46.56	50.5	41.6
9	109.4	130.	97.	97.03	120.	79.9	72.74	86.7	61.3	65.7	87.5	45.2	60.7	74.	50.	42.81	48.1	38.4
12	114.18	137.	95.	95.58	104.	86.1	70.45	82.5	61.2	62.75	75.5	53.9	51.07	67.6	46.1	40.36	48.5	34.6
Slaked in 200% of Water																		
3	60.8	72.	46.5	54.83	68.	44.8	54.68	61.2	42.9	44.22	45.	37.	50.04	55.9	43.7	41.62	45.6	35.9
6	55.21	66.2	38.	59.22	63.1	54.4	51.8	56.	43.	43.12	52.	38.	45.31	52.	39.1	43.79	50.	35.6
9	61.56	70.2	51.5	56.64	64.6	50.	48.	54.7	41.6	38.10	44.7	28.1	37.9	45.	30.	35.60	40.8	21.3
12	57.05	64.	48.8	56.74	67.3	49.	55.5	62.5	48.9	45.22	58.3	36.4	38.7	44.6	29.1	37.97	46.6	25.2
Slaked in 300% of Water																		
3	40.6	50.1	33.2	44.71	54.2	35.1	53.27	59.1	42.9	48.35	54.6	41.	46.	51.5	40.	47.92	56.7	48.1
6	45.68	50.4	33.2	49.37	59.4	38.5	51.13	58.5	42.4	46.94	57.	40.	43.65	51.	38.5	45.79	51.	37.1
9	44.9	52.6	35.8	45.25	58.5	38.	50.69	54.9	45.1	48.53	55.1	44.7	38.94	45.	29.2	50.50	54.8	47.1
12	42.55	53.2	22.7	50.25	57.8	35.7	62.58	73.6	49.4	49 03	57.7	43.2	39.37	46.9	25.5	45.01	48.5	40.2
Slaked in 400% of Water																		
3	64.07	74.4	55.8	62.9	68.	56.9	75.7	92.3	68.	75.67	87.8	68.2	68.01	79.	58.9	58.01	66.6	49.6
6	71.07	83.4	52.1	69.5	84.	46.	75.33	90.4	67.6	69.5	79.4	61.7	55.3	59.2	49.1	59.05	64.1	52.8
9	59.4	65.9	51.	60.74	74.6	46.2	60.9	72.	50.	63.02	67.9	57.5	46.42	54.2	32.7	47.98	54.9	34.3
12	74.43	84.	63.8	72.61	83.2	60.4	77.32	87.	71.				49.74	53.9	42.3	54.67	61.2	45.6



Sheet 4. White Lime from Springfield, Mo.



Sheet 5. White Lime from Springfield, Mo.



Sheet C. White Lime from Springfield, Mo.

5) are fairly uniform, all indicating a decrease in strength to two hundred or three hundred per cent followed by a rise with four hundred per cent of water. This final rise seems erratic, and its meaning is not at present understood. It will be noted that the greater ranges between highest and lowest tensile strengths are found where the lower sand-lime ratios were employed.

Sheet 6 brings out clearly the decrease in strength with increase in the amount of sand. This is most prominent in the lower percentages of water where a 1:1 mixture is the strongest. With the two higher percentages, three hundred and four hundred, the average maximum strength is attained in the 3:1 and 4:1 mixtures. It is also conspicuous that the highest figures of all are reached with the lime when slaked with one hundred per cent of water which leaves it a dry powder.

MAGNESIAN AND DOLOMITIC LIMES.

Eagle Point, Iowa, Brown Lime.

The limestone from which this lime is manufactured comes from the Galena beds of the Ordovician. Its analysis is as follows:

Insoluble	8.65
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	3.15
Lime (CaO)	29.00
Carbon dioxide (CO_2)	22.60
Magnesia (MgO)	17.12
Carbon dioxide (CO_2)	18.85
Water	0.09
	99.46

Analysis of the commercial product:

Insoluble	2.01
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	6.60
Lime (CaO)	58.19
Magnesia (MgO)	33.48
Loss on ignition	Slight
	100.28

The Eagle Point lime slakes sluggishly and gives a paste of brownish color. With all percentages of water employed there was little heating up and no steam was generated. Slaking proceeded best with two hundred per cent of water and more rapidly than with lower percentages. One hundred per cent gave a stiff paste, two hundred one of a readily and conveniently workable consistency, while the three hundred per cent paste was thin, and required the removal of the excess of water before use.

Table III includes the results of the tests of this lime. On curve sheets 7, 8 and 9 are plotted the data of the table.

The curves on sheet 7 again bring out the decrease in strength with age. It will be observed that the maximum strengths are attained at six and nine months, and that this almost universally falls off for the one year period. This falling off is most pronounced in general in the case of the higher sand proportions.

The influence of the amount of water used in slaking is shown on Sheet 8. It is impossible to make any generalized statements from the diagrams. As a rule the highest strengths are found with the lower percentages of water. In the case of the 2:1, 5:1 and 6:1 sand-lime ratios, however, this is reversed and the three hundred per cent gives the highest figures.

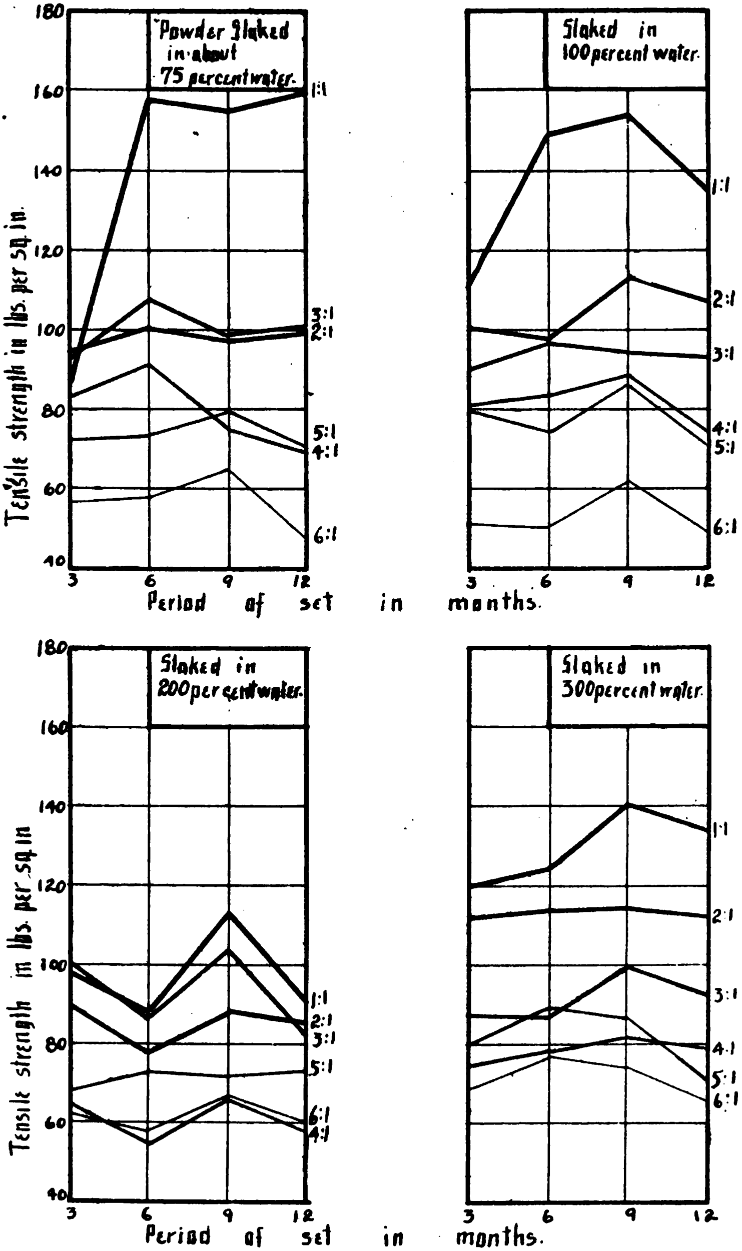
As with the white limes, the curves based on the sand-lime ratio are the most characteristic of the set. The lowest proportions of sand gave in all instances the highest results. The decrease in strength with increasing sand is decided and rapid. The greatest range is seen to be with the lower percentages of water, the maximum tensile strength being shown by the 'powder' slaked lime.

TABLE III

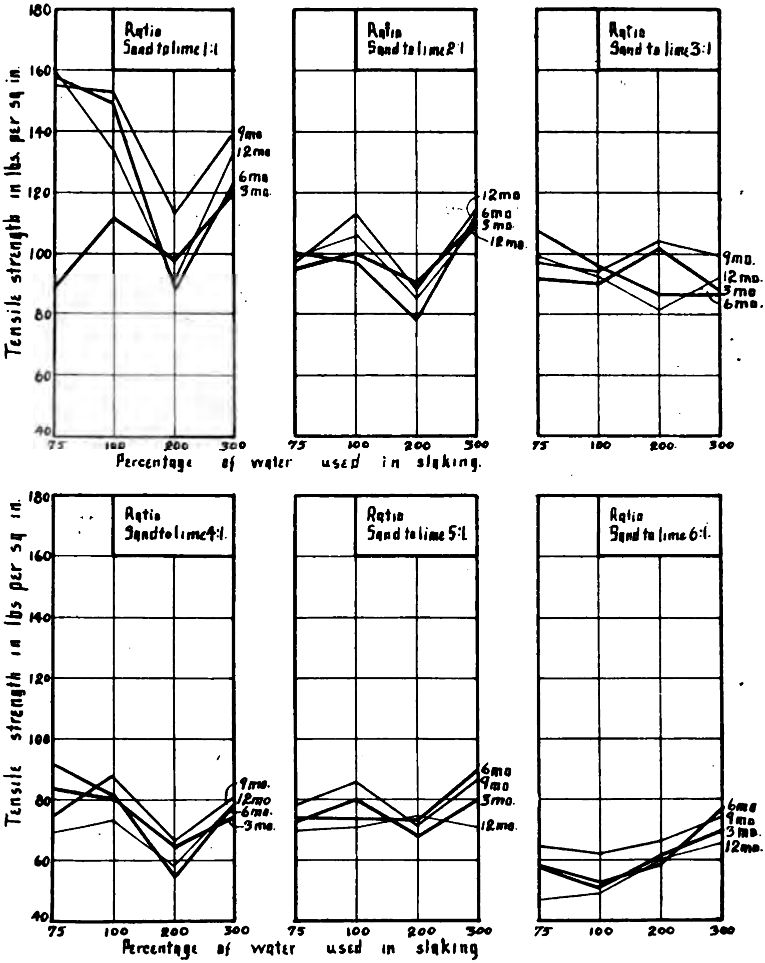
EAGLE POINT BROWN LIME

"Powder" Slake About 75% of Water

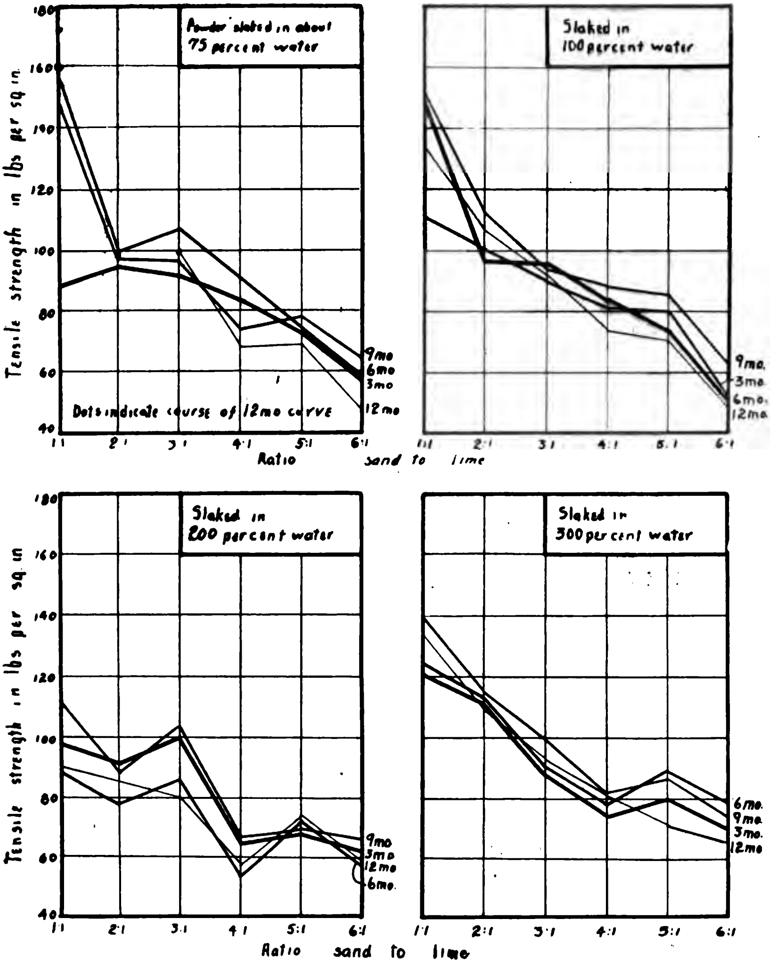
Time of Set in Months	Tensile Strength in Pounds Per Square Inch																	
	Ratio Sand to Lime 1:1			Sand to Lime 2:1			Sand to Lime 3:1			Sand to Lime 4:1			Sand to Lime 5:1			Sand to Lime 6:1		
	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.
3	88.85	120.	73.	94.9	110.	80.	90.92	104.8	66.7	84.26	98.	74.	72.60	82.5	60.	57.3	61.	55.8
6	157.9	176.	130.	99.6	125.	75.	106.67	116.2	95.2	90.97	104.9	80.	73.32	92.8	63.9	57.5	66.	50.
9	155.3	181.2	130.1	96.5	105.	82.	96.95	107.6	74.3	74.89	81.9	66.7	78.62	93.1	66.3	64.78	77.	52.
12	159.03	182.8	120.1	99.1	119.7	76.5	98.82	112.7	89.5	68.91	92.1	49.5	69.90	84.8	57.6	47.39	56.3	34.3
Slaked in 100% of Water																		
3	110.7	135.	85.	100.04	110.	82.5	90.41	100.	80.	80.67	96.8	60.	80.4	96.	70.	50.7	55.	40.
6	148.54	169.4	116.3	96.51	103.9	75.	95.9	122.	84.	82.6	93.	68.	74.28	87.6	47.6	50.43	56.4	39.6
9	154.08	208.2	119.7	113.31	132.	97.1	94.38	110.	71.4	87 9	104.9	72.8	86.2	98.4	63.7	62.2	72.8	52.4
12	135.38	159.4	105.2	107.32	127.5	89.6	93.08	110.1	80.2	73.85	92.2	50.5	70.71	88.1	63 1	49.10	58.4	39.6
Slaked in 200% of Water																		
3	97.7	115.	70.	89.9	109.	72.	100.6	105.9	95.1	64.6	73.5	61.	67.9	79.2	54.5	62.31	75.	53.
6	87.54	97.	73·3	78.11	92.7	68.6	85.94	97.2	62.5	55.1	62.5	40.4	73.27	87.4	58.8	57.72	62.1	48.5
9	112.66	124.5	94.1	87.96	110.	85.	104.05	113.2	89.6	66.4	75.5	58.8	72.48	87.4	59.2	65.8	75.	55.7
12	90.46	107.2	73.5	85.31	99.	75.	80.88	90.4	67.3	57.7	65.7	50.5	73.38	84.4	60.2	59.58	67.	52.4
Slaked in 300% of Water																		
3	119.85	145.	101.1	112.16	126.3	93.7	86.8	98.	70.	73.57	81.6	70.	80.1	100.	68.	68.87	77.	60.
6	124.25	142.4	103.	113.21	132.4	94.2	86.5	101.	74.3	78.31	92.4	70.5	88.92	101.	77.7	78.12	84.2	70.3
9	140.24	160.	125.	114.12	126.6	100.1	99.19	120.	79.1	81.24	92.6	71.1	87.02	95.1	75.7	74.25	90.2	56.
12	134.40	152.	105.9	111.95	119.2	101.	91.6	98.	75.7	79.17	95.	66.	70.71	88.1	50.5	66.16	75.2	58.4



Sheet 7. Dolomitic Lime from Eagle Point, Iowa.



Sheet 8. Dolomitic Lime from Eagle Point, Iowa.



Sheet 9. Dolomitic Lime from Eagle Point, Iowa.

MASON CITY, IOWA, BROWN LIME.

The Mason City stone is Middle Devonian and belongs to the Cedar Valley stage. Its chemical analysis follows:

Insoluble	1.34
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	2.07
Lime (CaO)	33.54
Carbon dioxide (CO_2)	26.35
Magnesia (MgO)	16.99
Carbon dioxide (CO_2)	18.68
Moisture	1.03
	100.00

Analysis of the commercial product:

	Quick-lime.	After slaking.
Insoluble	2.32	0.80
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	6.03	9.80
Lime (CaO)	72.40	47.60
Magnesia (MgO)	15.23	19.20
Carbon dioxide (CO_2)	0.10	3.50
Loss on ignition	3.36	18.70
Totals	99.44	99.60

The Mason City lime slakes very slowly and considerable care was necessary to secure a uniform product free from lumps. By a proper adjustment of the amount of water supplied as it is needed, and its distribution throughout the mass by stirring, a homogeneous paste is however readily obtained.

In Table IV are arranged the data for this lime, which are also plotted on curve sheets 10, 11 and 12.

The curves on Sheet 10 show for the Mason City lime the same tendencies as do the corresponding curves on preceding pages. The diminution in strength with the longest time period, while not universal, is so common as to be unmistakable. This is more pronounced in cases where the higher proportions of sand are used.

Sheet 11 clearly shows the decrease in strength with increasing percentages of water. With few exceptions the maximum results come with the powder slaked lime from which the curves slope downward as the water percentages increase.

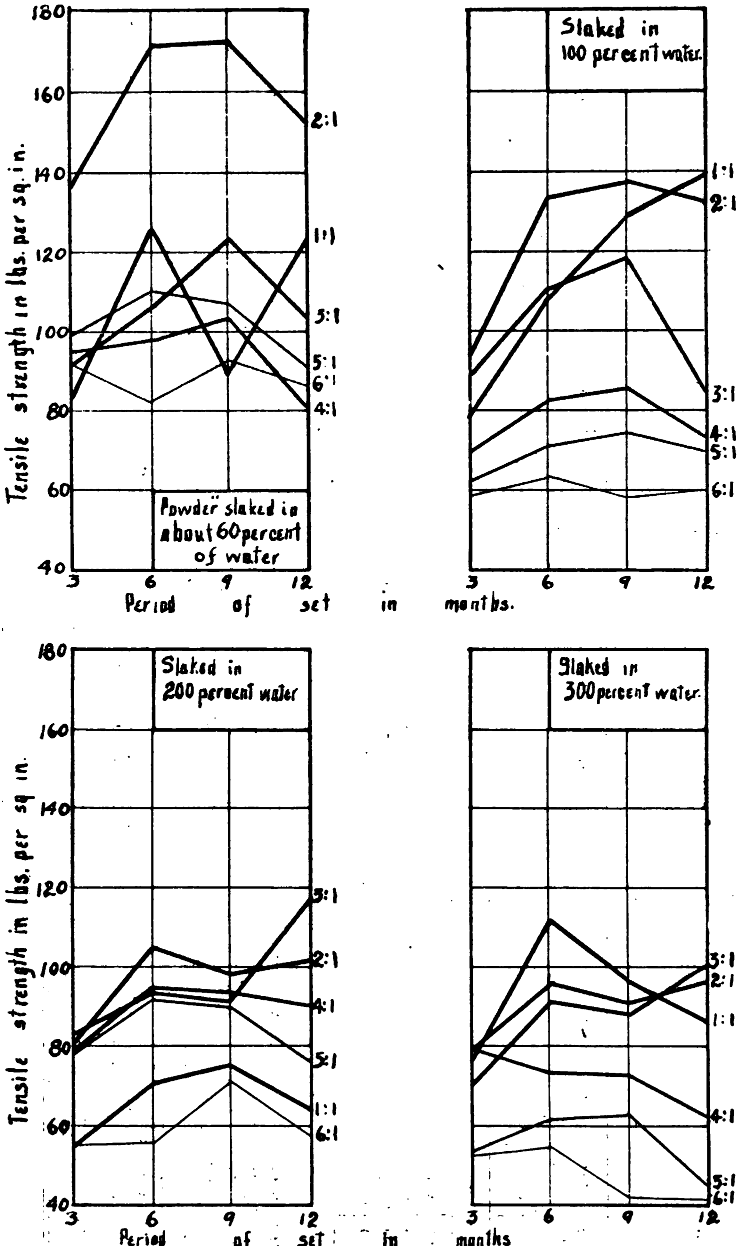
It will be noted also that the higher strengths correspond with the lower sand dilutions. With the powder slaked lime there is a decisive rise in all the curves at the 2:1 ratio and this is less prominently the rule with the other curves on the sheet. The length of the setting pe-

TABLE IV

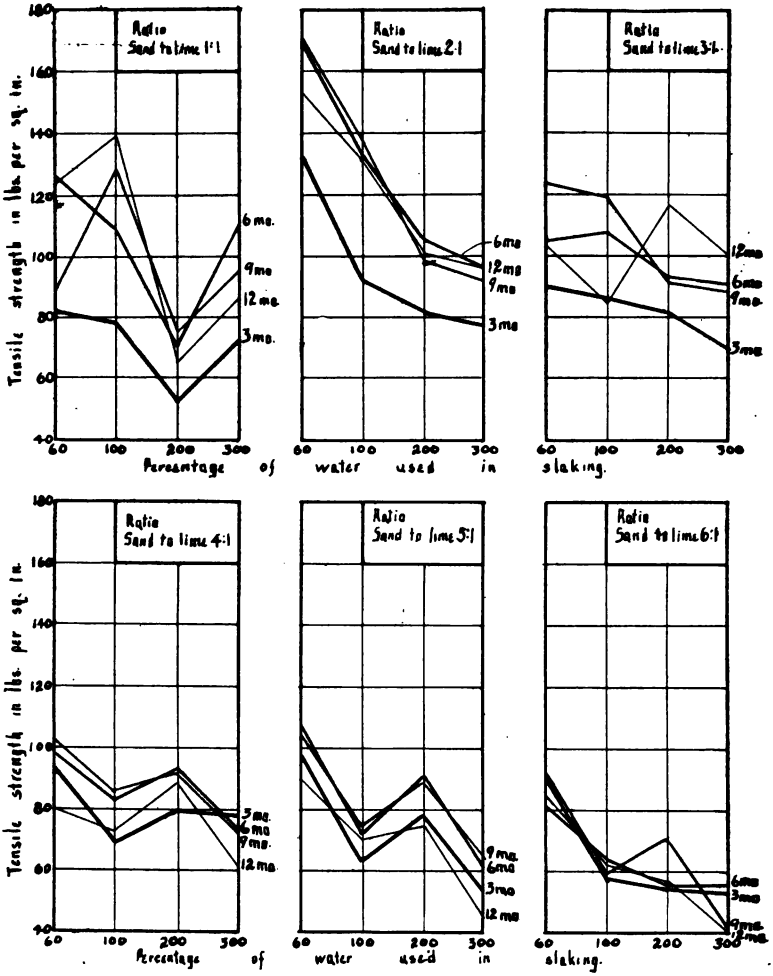
MASON CITY BROWN LIME

"Powder" Slaked—About 60% of Water

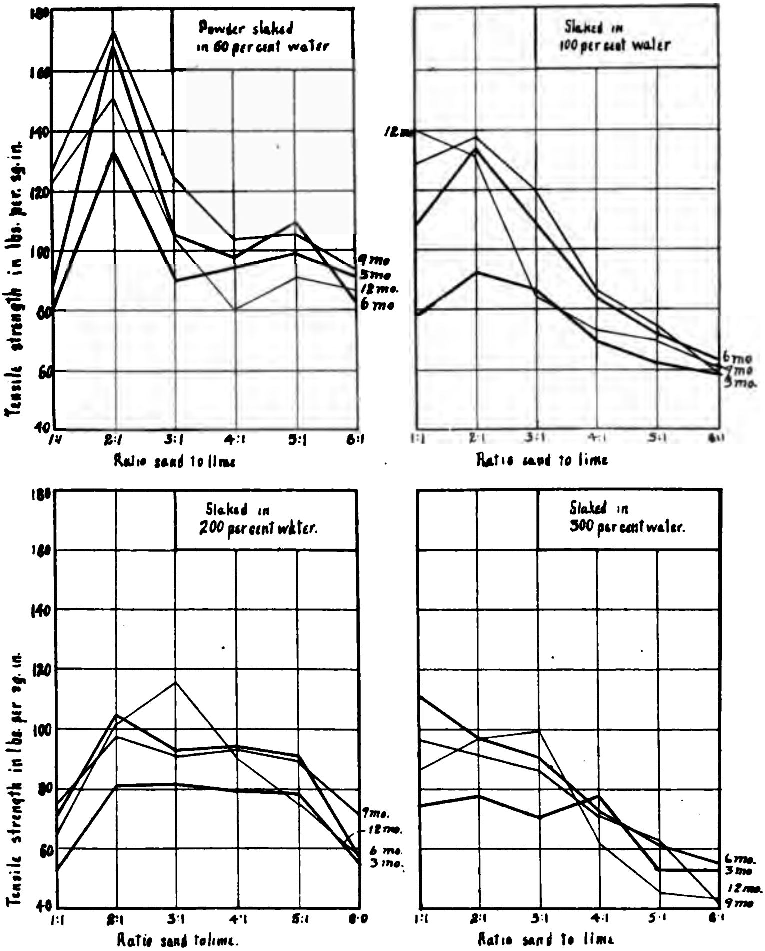
Time of Set in Months	Ratio Sand to Lime 1:1			Sand to Lime 2:1			Sand to Lime 3:1			Sand to Lime 4:1			Sand to Lime 5:1			Sand to Lime 6:1		
	Av. Tensile Strength lbs. per sq. in.	Max. T. S. lbs. per sq. in.	Min. T. S. lbs. per sq. in.	Av. Tensile Strength lbs. per sq. in.	Max. T. S. lbs. per sq. in.	Min. T. S. lbs. per sq. in.	Av. T. S. lbs. per sq. in.	Max. T. S. lbs. per sq. in.	Min. T. S. lbs. per sq. in.	Av. T. S. lbs. per sq. in.	Max. T. S. lbs. per sq. in.	Min. T. S. lbs. per sq. in.	Av. T. S. lbs. per sq. in.	Max. T. S. lbs. per sq. in.	Min. T. S. lbs. per sq. in.	Av. T. S lbs. per sq. in.	Max. T. S. lbs. per sq. in.	Min. T. S. lbs. per sq. in.
3	82.24	97.1	60.	134.35	182.5	110.6	90.22	96.1	77.4	94.15	108.9	86.5	98.9	109.8	86.5	90.49	110.6	75.7
6	125.57	165.	80.	170.92	184.	156.	104.56	117.	90.	97.5	109.	82.	110.37	122.	98.	82.30	90.	71.
9	89.4	128.	50.	172 45	186.3	150.	123.48	144.2	100.	103 16	122.5	91.2	106.61	122.	90.3	91.72	101.9	79.9
12	123.98	146.5	98.	151.58	168.	136.2	102.73	129.3	83 8	79.65	88.	65.	90.9	107.	42.	86 45	104.	76.2
Slaked in 100% of Water																		
3	77 83	95.6	50.	91.91	107.1	79.2	85.84	98.	77.7	68.62	83.3	58.	61.86	68.	53.	57 55	69 3	31.3
6	108.14	161.1	79.5	133.23	150.	93.7	107 87	134 7	80 6	83.45	103.	57.7	71.44	84.	58.	63.31	83.	53.5
9	159.26	158.1	102.	137.47	156.9	112.	118.70	144.	95.1	85.39	98.	68.6	73.64	96.9	54.1	57.93	67.	51.5
12	138.72	166 3	112.2	131.9	150.	107.	84.29	108.8	64.7	72.5	93.	60.	70.45	80.2	61.4	60.41	77.4	49.5
Slaked in 200% of Water																		
3	53.64	81.	30.	80.6	102.3	64.3	81.4	96.1	74.3	78 98	91.1	66.	78.49	90.3	59.4	55.4	65.	48.
6	69 63	95.3	49.8	104.72	121.8	94.1	93 3	112.	75.	94.	120.	75.	92.2	103.	81.	56.39	73.7	40.
9	74.7	88.5	63.5	97.78	127 5	84.7	90.98	113.8	73.2	93.35	117 1	71.5	90.8	102.	75.	71.4	85.	59.
12	64.49	78.1	46.8	100.91	121.	70.8	116.64	131.	99.	89.82	99.	77.8	75.91	92.8	60.6	57.79	68.	49.5
Slaked in 300% of Water																		
3	74.66	93.	58.8	77.01	87.	64.	70.44	78.4	59.6	77.69	88.2	64.4	53.23	58.8	44.1	52.99	60.	40.
6	110.65	129.6	85 7	95.64	110.	75.	91.3	104.	72.	73.3	80.	63.	60 56	77.	50.	55.46	65.7	45.
9	96.01	104.9	93.	90.66	100.	74.5	88.11	111.7	68.6	71.65	81.3	63.7	62.30	74.5	52.	40.59	48.1	29.4
12	85.75	103.	64.6	95.99	116.	75.	100.19	119.6	87.2	61.8	76.	51.	45.2	54.	40.	40.52	48.5	29.7



Sheet 10. Dolomitic Lime from Mason City, Iowa.



Sheet 11. Dolomitic Lime from Mason City, Iowa.



Sheet 12. Dolomitic Lime from Mason City, Iowa.

riod appears to have no perceptible influence on this fact.

MAQUOKETA WHITE LIME.

A. A. Hurst & Co., Maquoketa, Iowa, Dolomitic Lime.

EXCELSIOR WHITE LIME.

O. W. Joiner & Son, Maquoketa, Iowa, Dolomitic Lime.

The Maquoketa limes are produced from the Hopkinton beds of the Niagara stage. The composition of these strata is shown by the accompanying chemical analyses of samples from each of the companies:

	O. W. Joiner & Son Per cent.	A. A. Hurst & Co. Per cent.
Insoluble	0.51	0.58
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	0.47	0.36
Lime (CaO)	30.56	30.88
Magnesia (MgO)	21.54	21.56
Loss on ignition (CO_2 and water)	47.16	47.13

Analysis of the unslaked commercial product (A. A. Hurst & Co.):

Insoluble	0.63
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	2.10
Lime (CaO)	60.60
Magnesia (MgO)	35.70
Loss on ignition	2.30

The two samples of limestone analyzed come from the same horizon and compare very closely in composition. The treatment of the rock in the process of burning is exactly similar in both plants and the two limes are alike in appearance. Wood alone is used in the calcining process.

The quick-limes slake slowly, as is characteristic of the dolomitic limes, and the heat generated is relatively small in amount. The Joiner lime required somewhat less water for the first slaking than the Hurst product and, in fact, less than any of the other limes tested. This quantity, as shown in the table, is fifty per cent of the weight of quick-lime used.

The results of the tensile strength tests of the two Maquoketa limes are compiled in tables Nos. V and VI and are graphically represented by curve sheets 13, 14,

TABLE V

EXCELSIOR WHITE LIME, MAQUOKETA

"Powder" Slaked in About 50% of Water

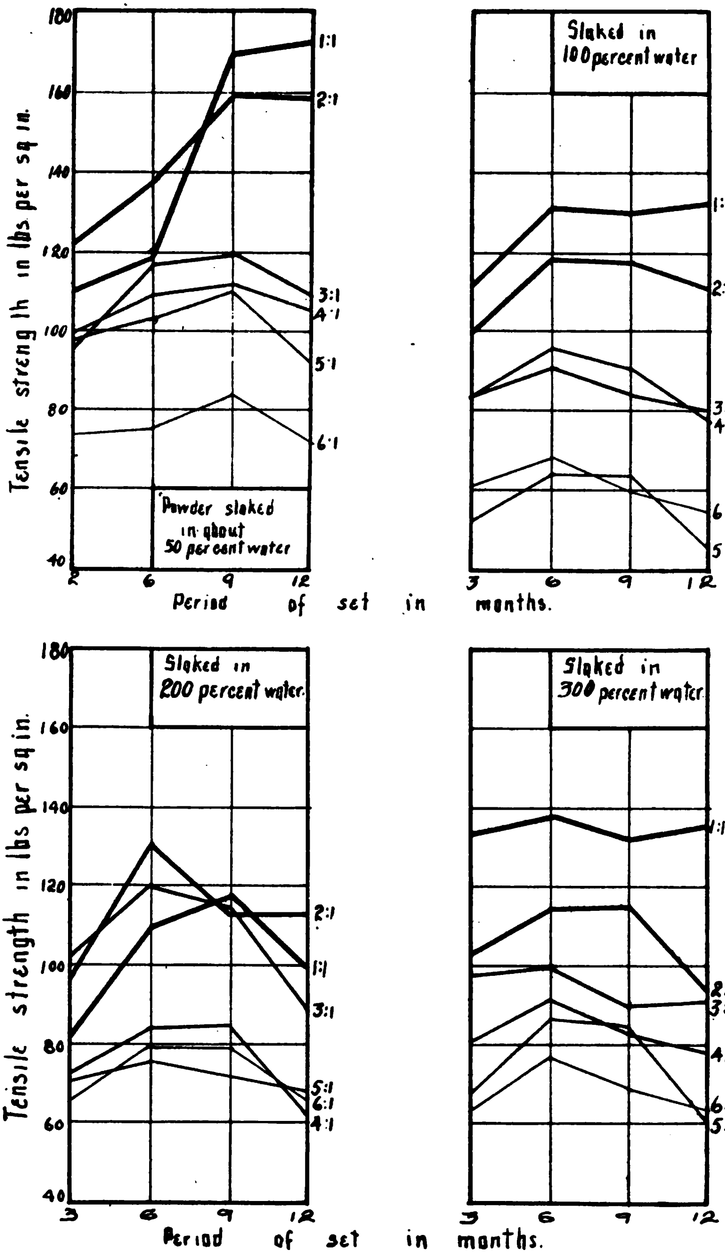
Time of Set in Months	Trusile Strength in Pounds Per Square Foot																	
	Ratio Sand to Lime 1:1			Sand to Lime 2:1			Sand to Lime 3:1			Sand to Lime 4:1			Sand to Lime 5:1			Sand to Lime 6:1		
	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.
3	109.68	136.	85.	121.82	140.	111.6	95.65	115.	78.4	98.9	115.7	81.	98.61	111.6	78.4	73.63	80.9	68.
6	118.12	134.3	100.	137.18	174.8	103.9	117.33	130.4	107.8	108.92	137.2	85.3	103.10	115.5	91.2	75.21	84.5	67.
9	169.61	198.	139.3	158.72	194.1	132.3	119.76	132.	100.	112.34	120.6	104.9	109.71	119.4	101.9	84.26	94.2	72.8
12	171.53	193.4	151.	158.36	173.5	141.6	108.56	124.3	98.2	105.4	116.	92.	90.94	108.8	58.8	72.14	84.3	63.7
Slaked in 100 % of Water																		
3	110.59	127.5	91.8	99.5	105.	93.3	83.5	97.	78.	83.59	95.1	65.	51.71	64.1	38.8	61.4	77.	50.
6	130.7	154.6	113.4	118.91	136.3	100.	91.01	100.	83.6	96.05	117.1	78.4	64.1	79.6	56.3	69.15	82.5	55.3
9	129.9	147.	114.	118.1	135.	102.	84.4	95.	65.	91.4	99.	86.	64.	69.	60.	60.3	71.6	52.
12	132.09	144.5	120.1	111.21	124.3	101.	80.21	93.9	58.6	78.17	84.1	65.3	45.24	57.3	34.6	54.74	64.4	45.5
Slaked in 200 % of Water																		
3	83.	95.	70.	97.02	115.	84.1	102.29	108.	95.1	72.66	85.	60.	70.9	78.4	55.9	65.47	74.	52.
6	109.5	133.	77.	130.89	150.6	118.5	119.84	134.6	104.7	83.85	98.	73.	75.82	80.6	74.2	78.86	98.	63.7
9	117.7	127.	96.	113.08	133.3	88.6	113.65	139.1	100.	84.8	100.9	67.2	72.3	76.9	67.3	79.5	100.	53.9
12	99.38	120.2	82.8	113.31	130.5	87.3	89.72	106.8	78.9	62.16	75.2	56.4	67.7	82.	44.	65.54	76.	57.4
Slaked in 300% of Water																		
3	134.33	157.3	117.8	102.71	122.4	85.	98.42	110.	91.	80.8	88.	73.	68.31	78.4	60.	63.31	73.5	52.9
6	138.32	168.2	111.7	115.49	136.	103.	99.67	107.9	88.3	91.6	107.	85.2	86.61	103.9	76.7	77.27	84.2	71.3
9	131.82	153.1	106.2	115.52	130.	95.	89.98	115.4	94.2	83.07	92.3	72.1	84.91	101.	69.2	69.49	81.7	57.7
12	134.87	147.	118.1	92.91	111.8	64.4	90.66	97.2	77.7	77.67	94.2	49.	60.89	71.3	49.5	62.59	69.9	48.5

TABLE VI

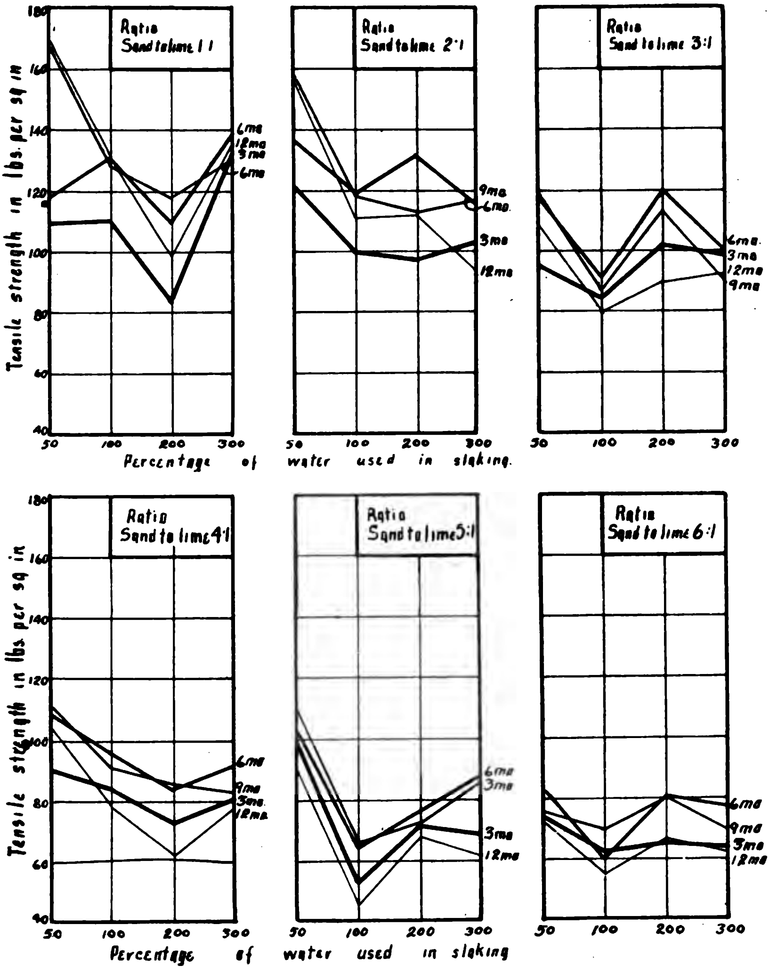
MAQUOKETA WHITE LIME

"Powder" Slaked—About 70% of Water

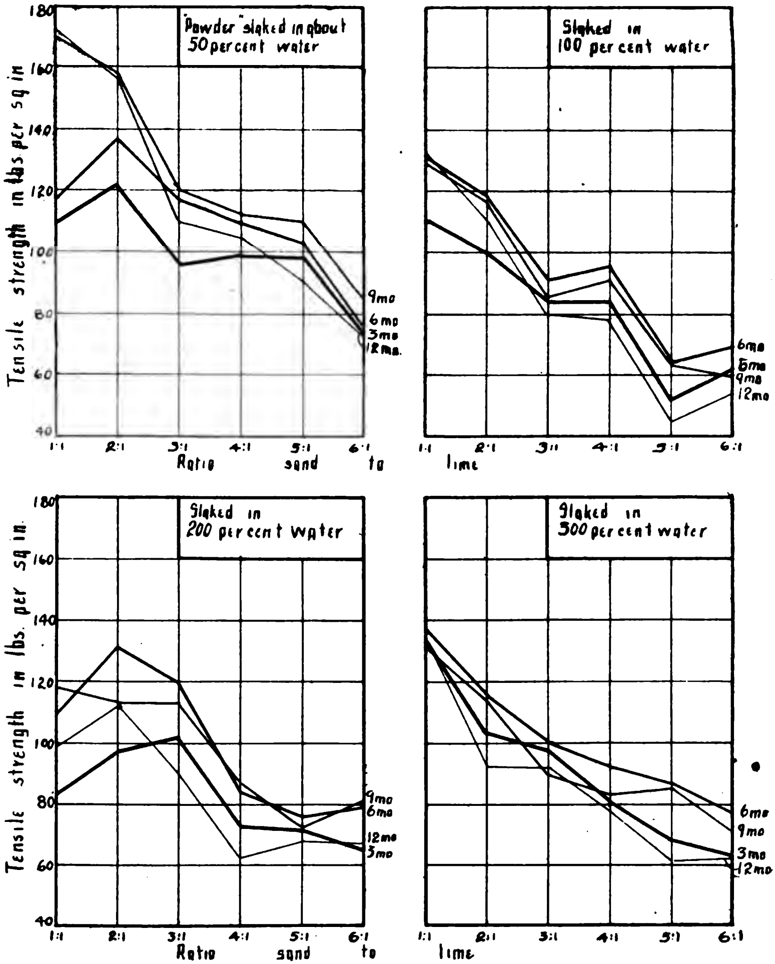
Time of Set in Months	Ratio Sand to Lime 1:1			Sand to Lime 2:1			Sand to Lime 3:1			Sand to Lime 4:1			Sand to Lime 5:1			Sand to Lime 6:1		
	Av. t. s. lbs. per sq. inch	Max. t. s. lbs. per sq. inch	Min. t. s. lbs. per sq. inch	Av. t. s. lbs. per sq. inch	Max. t. s. lbs. per sq. inch	Min. t. s. lbs. per sq. inch	Av. t. s. lbs. per sq. inch	Max. t. s. lbs. per sq. inch	Min. t. s. lbs. per sq. inch	Av. t. s. lbs. per sq. inch	Max. t. s. lbs. per sq. inch	Min. t. s. lbs. per sq. inch	Av. t. s. lbs. per sq. inch	Max. t. s. lbs. per sq. inch	Min. t. s. lbs. per sq. inch	Av. t. s. lbs. per sq. inch	Max. t. s. lbs. per sq. inch	Min. t. s. lbs. per sq. inch
3	112.6	135.	70.	143.87	157.7	117.5	121.58	133.	115.4	105.29	120.	100.	104.74	130.	85.	98 39	115.7	88.2
6	140.05	197.	100	213.52	243.9	178.2	125.	141.	102.	115.38	133.6	97.	126.38	142.9	108.1	110.81	125.	93.1
9	159.38	176.7	145.2	191.56	214.8	170.1	139.2	139.2	110.8	131.45	144.1	124.5	113.53	138.8	90.7	102.47	115.7	90.2
12	150.03	180.6	121.8	182.98	210.	158.	139.89	147.5	108.9	122.16	146.5	96.	118 26	138.6	78.4	104.70	118.	87.
Slaked in 100% of Water																		
3	128.43	144.	110.	130.14	153.	110.	130.	140.	120.	115.93	126.2	100.	101.64	111.7	86.5	97.66	110.6	82.5
6	178.95	204.1	151.	156.75	177.	144.9	143.36	173.4	120.	134.16	158.4	120.	123.92	142.2	109.8	105.92	121.6	98.
9	184.29	213.2	150.	167.36	183.6	146.9	154.3	168.	137.	144.5	162.	111.	137.56	152.4	125.7	119.94	140.6	105.8
12	185.85	212.2	164.8	157.91	170.	130.9	144.47	161.6	132.3	123.05	133.7	94.9	128.16	137.4	117.1	108.1	116.	96.
Slaked in 200% of Water																		
3	105.01	120.	80.	105.09	136.5	89.1	99.82	121.	88.	91.04	110.6	81.7	103.62	120.	85.	81.72	90.	74.8
6	134.66	163.2	117.	125.20	151.5	101.	108.18	125.7	93.9	96 85	115.7	73.5	106.53	117.3	93.1	83.4	100.9	65.7
9	152.84	173.4	131.6	124.70	141.6	96.8	108.3	120.	93.	94.54	115.	84.5	85.26	95.2	77.7	92.1	99.	82.7
12	146.05	164.9	121.6	114.51	145.3	89.7	102.34	111.2	93.9	101.	112.1	87.9	97.18	118.2	70.7	95.19	112.1	82.
Slaked in 300% of Water																		
3	100.7	125.	90.	86.49	94.1	70.	82.25	95.	63.7									
6	126.8	134.	115.	98.07	117.	84.2	95.50	113.8	89.1									
8	123.22	136.7	113.2	112.6	136.	95.	95.90	108.	85.2									
12	124.9	138.1	113.0	99.27	131.9	79.3	91.91	114.3	78.6									



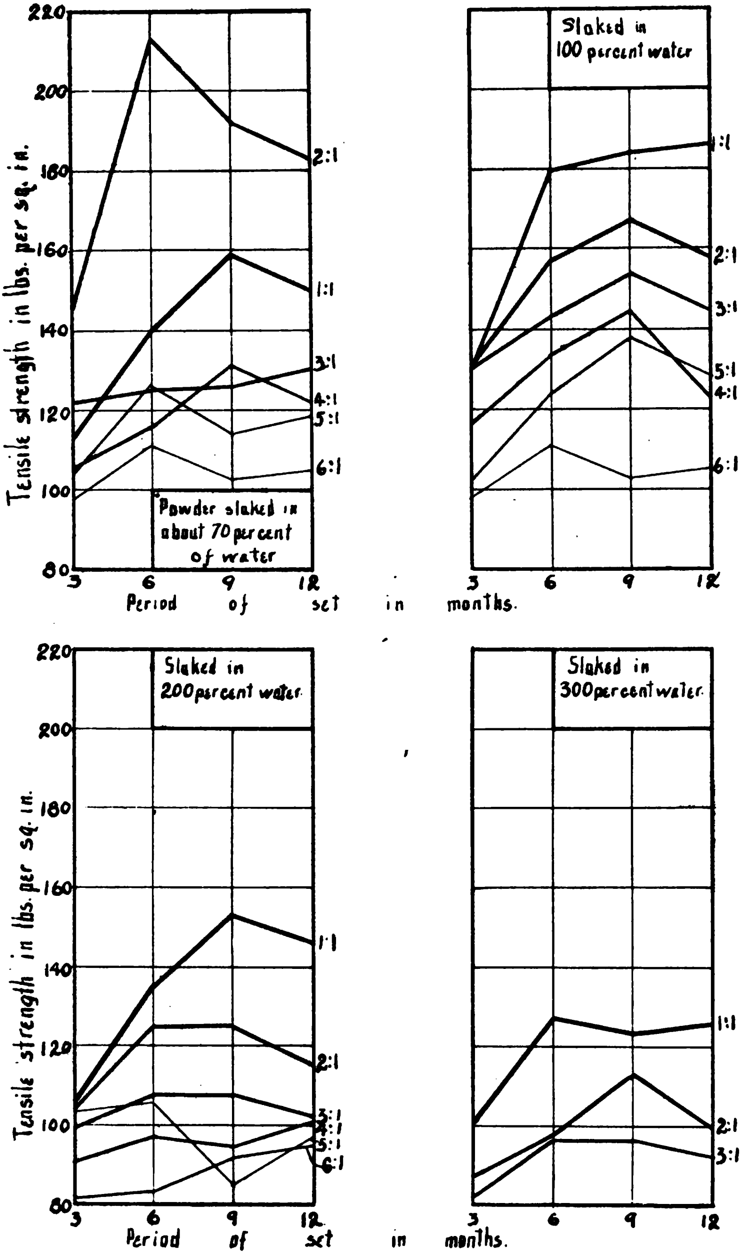
Sheet 13. Dolomitic Lime, O. W. Joiner & Son, Maquoketa, Iowa.



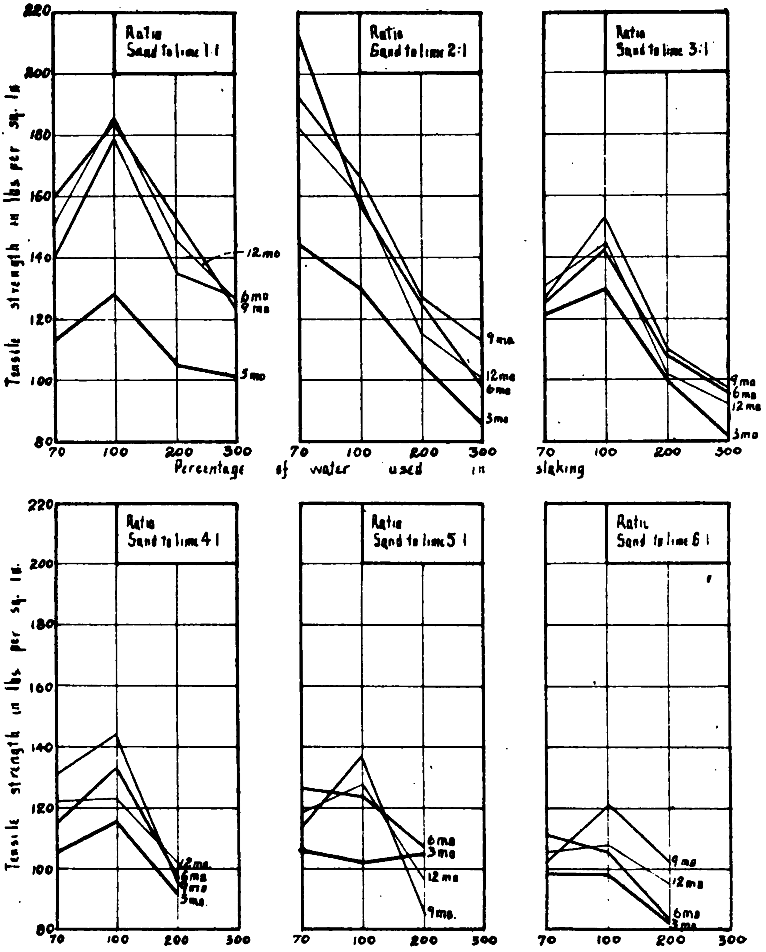
Sheet 14. Dolomitic Lime, O. W. Joiner & Son, Maquoketa, Iowa.



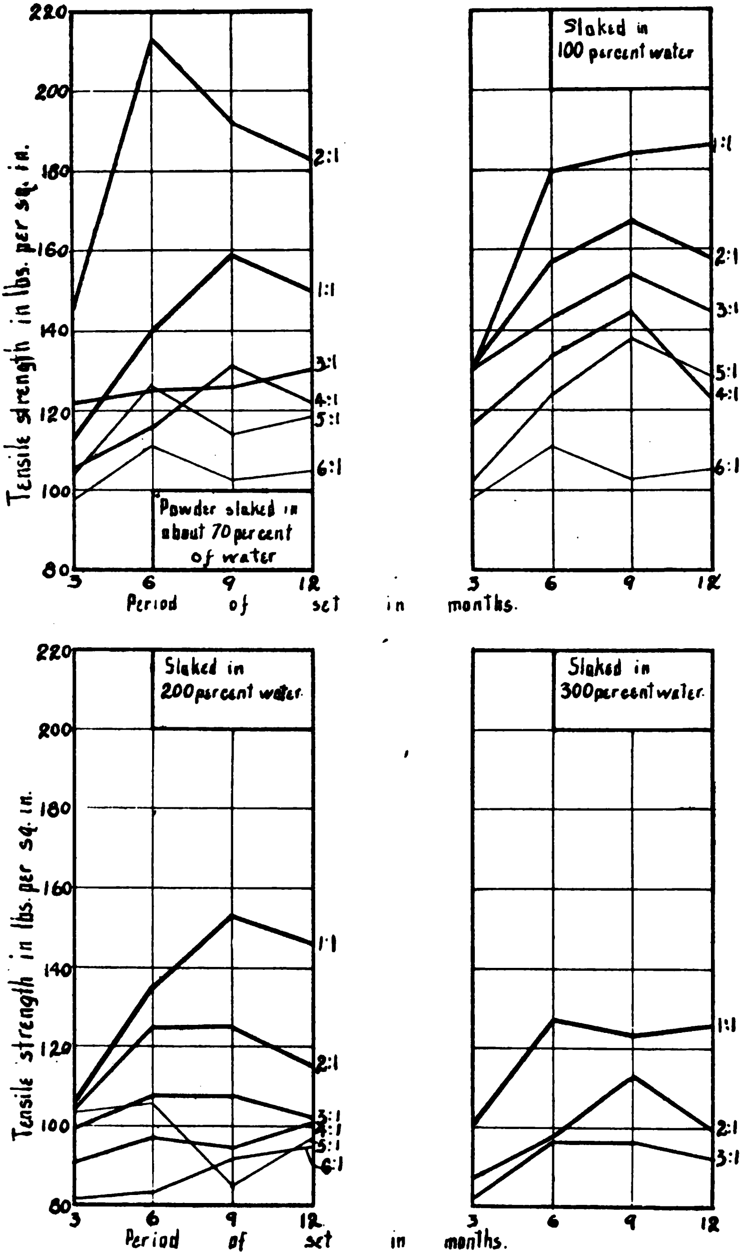
Sheet 15. Dolomitic Lime, O. W. Joiner & Son, Maquoketa, Iowa.



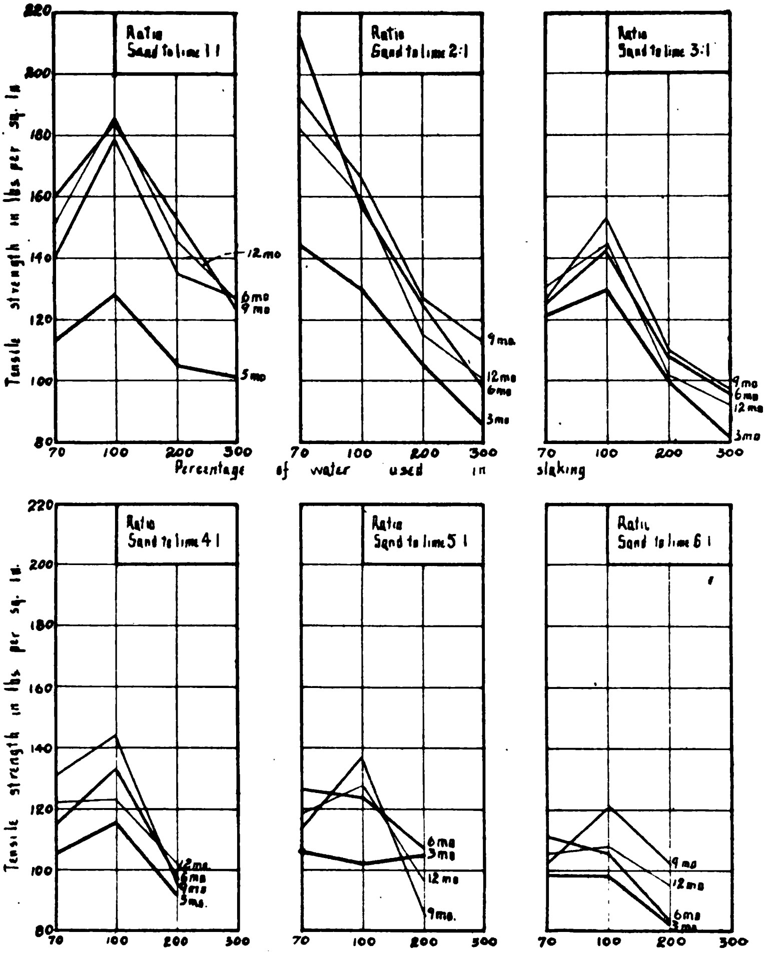
Sheet 16. Dolomitic Lime, A. A. Hurst & Co., Maquoketa. Iowa.



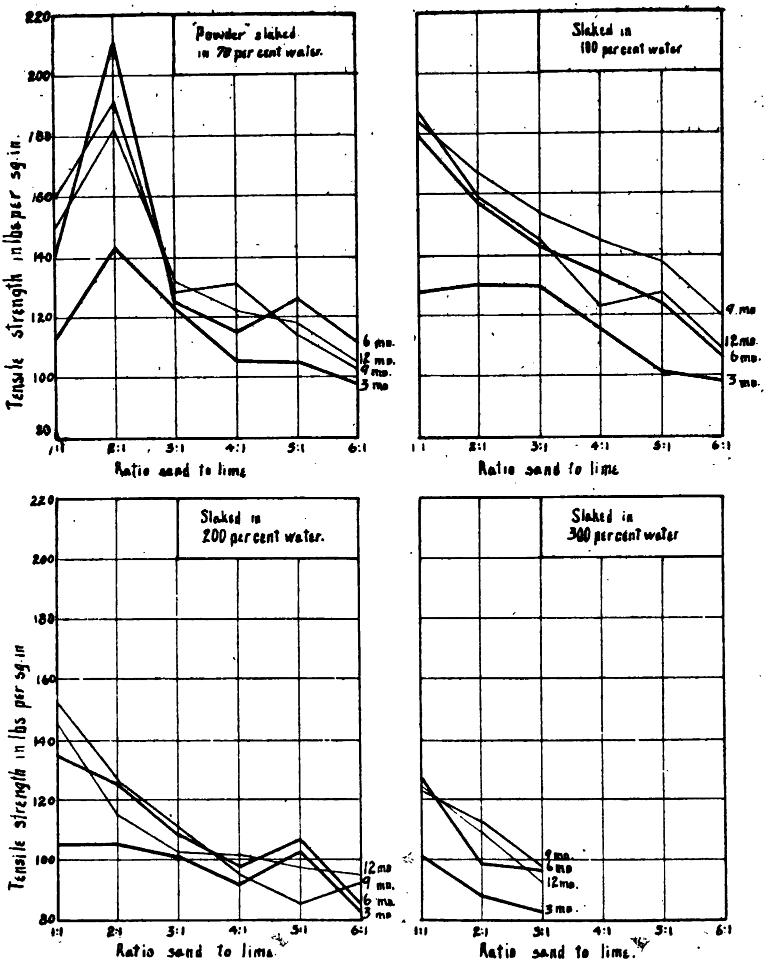
Sheet 17. Dolomitic Lime, A. A. Hurst & Co., Maquoketa, Iowa.



Sheet 16. Dolomitic Lime, A. A. Hurst & Co., Maquoketa. Iowa.



Sheet 17. Dolomitic Lime, A. A. Hurst & Co., Maquoketa, Iowa.



Sheet 18. Dolomitic Lime, A. A. Hurst & Co., Maquoketa, Iowa.

15, and 16, 17, 18 which follow.

Joiner lime: Sheet 13 brings out very well the change in strength with increasing period of set. The powder slaked lime attains its maximum strength at nine months with the marked exception of the 1:1 ratio in which case the curve continues upwards to the end of a year's time, and gives the highest figure of any mixture in the set. The other percentages of water give characteristic and fairly uniform curves, showing a maximum strength at six to nine months and a falling off on weakening after nine months' set.

The curves on Sheet 14 are somewhat irregular but exhibit quite clearly the decrease in strength with increase of water for slaking. In general, the slope is down from the 'dry' slake through all higher percentages, although in a number of instances there is an unaccounted for rise from the minimum at one hundred or two hundred per cent. The position of the curve groups on the diagrams indicates the lowering strength with larger sand dilutions.

Sheet 15 emphasizes the weakening effect of sand dilutions higher than 1:1 and 2:1 mixtures. In the majority of instances even proportions of sand and lime afford the greatest strength although a higher figure for 2:1 is not unusual. Higher ratios than these two, however, produce a marked falling off in tensile strength for all four time periods.

Hurst lime: The curves of Sheet 16 run conspicuously uniform and show in general, the greatest strength at nine months. The usual lowering in strength at the end of twelve months is to be noted. As a rule this lime attained its maximum strength with one hundred per cent of water as shown on Sheet 17. The highest figures of the set are reached, however, by the powder slaked lime and a sand-lime ratio of 2 to 1.

On Sheet 18 is brought out the relation between the strength and amount of sand used. With the powder slaked lime, 2:1 gives the highest results, while with the other percentages of water for slaking the trend of the curves is universally downward as the sand increases.

"NEW PROCESS LIME,"

Viola Lime Works, Viola, Iowa.

The Viola lime was manufactured from the Le Claire beds of the Niagara stage. The plant is now idle. The stone is highly magnesian and produces a lime of the following composition:

	Quick-lime.	After slaking.
Insoluble	1.20	1.00
Iron and aluminum oxides (Fe_2O_3 and Al_2O_3)	1.40	1.00
Lime (CaO)	66.80	45.20
Magnesia (MgO)	27.10	29.28
Carbon dioxide (CO_2)	.70	2.02
Loss on ignition less CO_2	2.50	21.75
	99.70	100.25

The commercial product takes water slowly and no slaking action becomes noticeable for some time. About five hours was required for complete slaking in sixty per cent of water, the mixture heating but slightly. With the higher percentages of water the time required is still greater and in all cases the slake is very cool. The lime does not melt to a paste as is usual, but remains in a more or less granular condition. The results of the tests of the Viola lime are tabulated in Table VII and plotted on curve sheets 19, 20 and 21.

A comparison of the data obtained in these tests with the results from the other limes of the whole series reveals two notable departures. The breaking strengths are on an average higher, the maximum being nearly fifty per cent greater than the closest competitive value. They are remarkable also in that the strength almost without exception increases to the end of twelve months and this increase is most rapid, as shown by the sharpness of the curves on Sheet 19, when slaked with one hundred per cent water which gives the highest breaking strength of the set. The steepness of the curves between nine and twelve months is in many instances so marked as to render of extreme interest the question, how long such increase would continue. A properly designed series of tests should be made along this line.

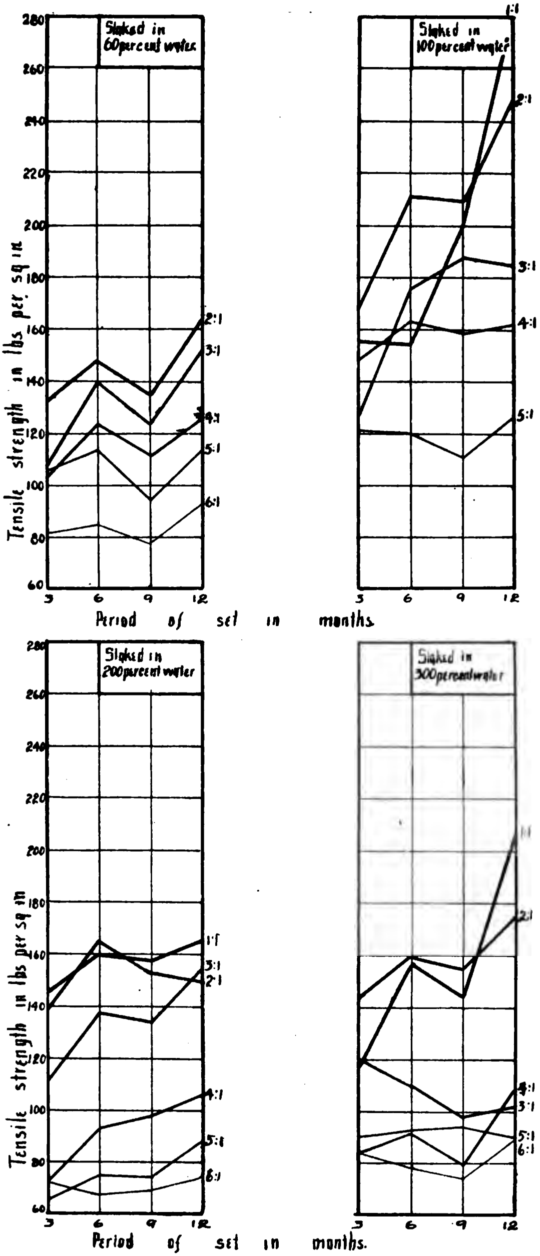
Sheet 20 shows that the lime develops its greatest

TABLE VII

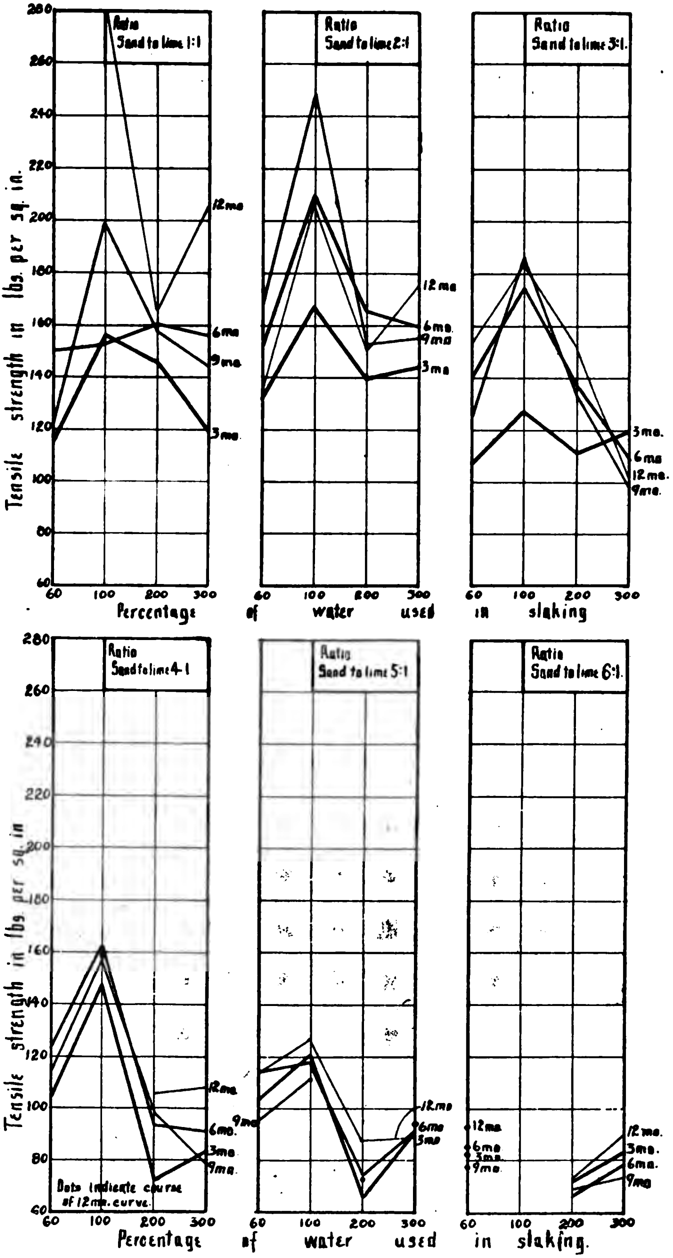
VIOLA NEW PROCESS LIME

“Powder” Slaked in About 60% of Water

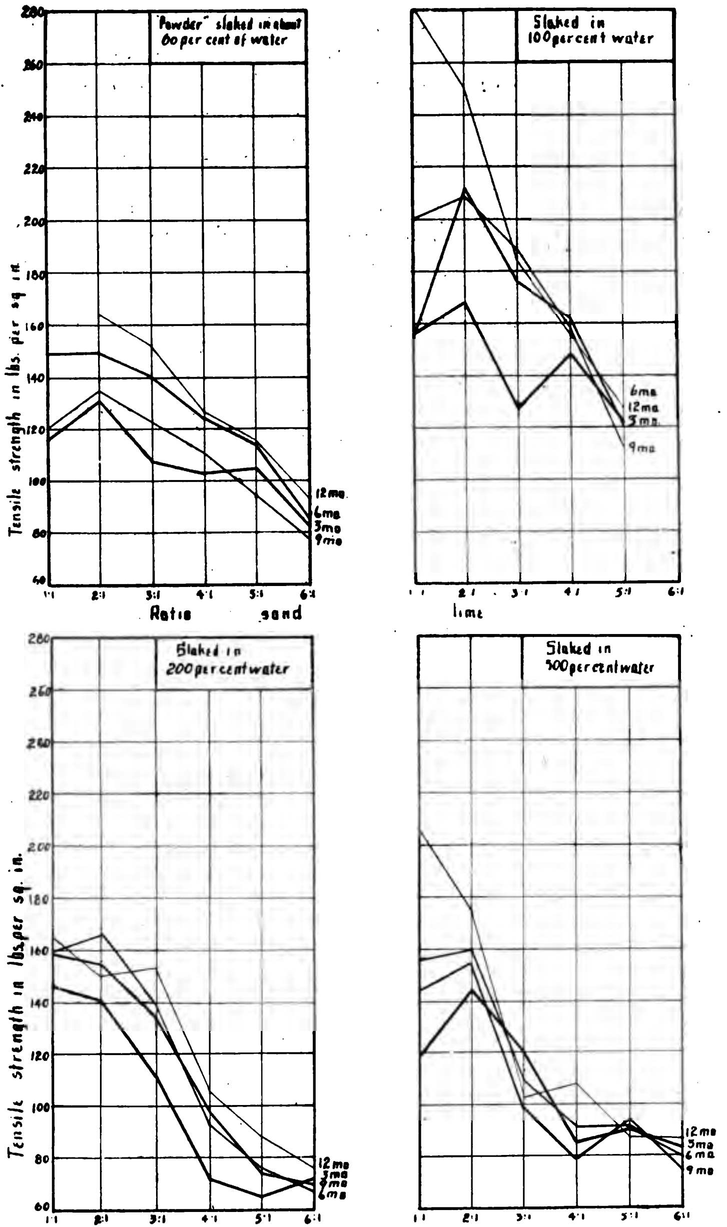
Time of Set in Months	Tensile Strength in Pounds Per Square Inch																	
	Ratio Sand to Lime 1:1			Sand to Lime 2:1			Sand to Lime 3:1			Sand to Lime 4:1			Sand to Lime 5:1			Sand to Lime 6:1		
	Av.	Max.	Min.	Av.	Max	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.
3	116.5	138.7	104.	130.8	152.	100.	107.2	122.6	89.6	101.9	113.4	92.3	103.7	120.7	88.	81.5	91.5	75.4
6	149.5	168.6	124.5	149.4	174.5	122.4	138.8	163.2	117.	122.9	152.8	102.9	113.9	134.2	90.7	84.6	95.4	74.1
9	119.6	142.1	95.2	134.8	157.7	110.3	122.4	150.	103.9	110.8	124.2	92.2	94.8	108.4	82.2	77.6	85.7	61.9
12				163.5	184.7	138.8	151.7	176.	120.2	125.2	140.3	107.6	113.6	122.6	102.8	93.3	107.7	83.7
Slaked in 100% of Water																		
3	156.4	199.	106.05	167.7	198.	132.	127.3	163.2	105.	148.4	180.6	116.5	120.7	140.1	102.8			
6	154.2	200.	132.3	211.4	244.9	176.5	175.4	208.	100.	163.1	212.	130.	120.3	143.5	92.6			
9	200.4	269.5	145.	209.2	253.	180.6	186.8	213.1	163.6	158.3	185.1	144.5	111.2	127.6	83.8			
12	282.1	301.9	255.6	249.7	278.2	191.6	183.9	206.1	165.9	161.6	178.4	151.9	127.2	150.	103.7			
Slaked in 200% of Water																		
3	145.7	185.4	103.	139.8	157.	112.	111.	124.2	95.1	72.1	100.9	70.5	65.1	71.4	57.1	72.1	89.4	63.4
6	160.4	201.	111.1	164.8	189.3	140.	137.7	182.5	107.	93.4	105.6	80.2	75.1	95.3	75.4	67.	77.6	56.1
9	157.9	185.9	121.3	153.4	173.5	142.6	134.	159.2	118.6	97.9	122.3	87.4	74.4	83.	66.3	69.1	77.6	60.6
12	165.1	189.5	142.7	149.6	164.6	127.3	153.1	176.5	128.9	105.7	118.4	90.2	88.4	102.8	73.6	74.3	83.8	65.7
Slaked in 300% of Water																		
3	118.4	143.	102.	143.8	159.6	114.3	119.7	132.7	106.3	84.1	90.7	77.8	90.4	107.6	60.	83.5	91.5	75.4
6	155.7	187.7	118.5	159.8	181.5	132.4	110.1	128.	86.9	90.8	104.8	78.2	91.3	103.8	76.9	78.7	87.7	70.7
8	144.1	182.3	94.6	154.5	166.7	133.2	97.9	114.	84.1	78.8	101.9	46.7	92.9	103.9	81.5	73.5	87.5	56.7
12	206.9	229.	165.	176.2	210.9	153.	101.2	128.4	85.9	107.7	119.2	83.7	90.	101.9	80.6	90.2	99.	76.9



Sheet 19. Dolomitic Lime, Viola Lime Works, Viola, Iowa.



Sheet 20. Dolomitic Lime, Viola Lime Works, Viola, Iowa.



Sheet 21. Dolomitic Lime, Viola Lime Works, Viola, Iowa.

strength when slaked in one hundred per cent of water, while on 21 is plainly shown the decrease in tensile strength following the addition of proportions of sand greater than one and two to one of lime.

GENERAL CONCLUSIONS.

While the foregoing series of tests are far from exhaustive and serve to open up and to suggest many questions that might be profitably investigated, the results obtained are, it is believed, sufficient to base upon them a few generalizations. Some of the statements which follow are of facts that have long been regarded in practice but which have not before been proven by systematic experiment. The limes tested are types of high grade products and may be regarded as representative examples of pure white limes and of the magnesian or dolomitic class. The results therefore, are to be limited in their general application to these classes and are not to be construed as holding good for the impure or hydraulic limes.

(a) The maximum strength reached within a year's time is attained at the end of a setting period of six to nine months duration. This is most pronounced where the higher percentages of sand are employed. The notable exceptions to this rule are found with the lowest sand-lime ratios, the lower percentages of water used to slake and are most conspicuous in the strongly dolomitic limes.

The cause for the diminution in strength after nine months is not known and results of chemical analyses to determine the amount of carbonation at the end of each of the four periods indicate that this process is in no instance complete at the end of a year's time. Carbonation has progressed to a minor extent only during the first six or even nine months. The change that occurs during the setting of the mortar is considered to be largely the crystallization of the lime hydrate. It is possible that such crystallization may produce a bond that is stronger than the carbonate. The process of carbonation displaces the combined water of the hydrate and may as a result actually weaken the cohesive strength of the mortar. If this be true, we would expect such loss in strength to continue till a minimum value is reached which would either remain constant or, as the amount of carbonate becomes greater than that of hydrate and carbonation approaches completion, increase again. It is within the range of probability that the ultimate final strength which might require years for attainment, would be greater than that reached in the first few months of setting. A set of long-time tests properly designed should yield valuable information along these lines.

(b) In general, the greatest strength comes with the lower percentages of water used in slaking. Equal amounts by weight of water and of dry quick-lime give in the majority of cases, the highest results. Higher proportions are detrimental to tensile strength. This is more especially noticeable in the white limes.

The generation of a considerable amount of heat, and consequently steam, seems essential in the slaking process, as explained earlier. Too little water leaves hydration and therefore expansion in bulk, incomplete and the unslaked lime remaining receives its necessary moisture either slowly from the atmosphere or from the water used in mixing for use. The later slow hydration is not accompanied by the necessary rise in temperature or increase in volume. Too much water prevents the formation of steam and maximum increase in bulk, and therefore retards the slaking. A high excess may keep the temperature so low that combination between water and quick-lime may be evidenced by few if any signs of slaking whatever, for hours after immersion. It would be expected, therefore, that such a percentage of water as would produce the most vigorous slaking action and leave a satisfactorily moist paste would afford the best results when tested. This amount varies with different limes as noted in the consideration of each set of results. In every instance, however, the percentage giving the highest strength was that amount which gave the best slake and produced the most workable paste.

(c) As a rule, the highest strength is given by the lowest proportions of sand, the curves being about equally divided between equal parts by weight of sand and dry quick-lime and two of sand to one of lime.

Economy in the use of lime demands that as little as possible be used over that required to fill the voids and to coat each grain of sand with a thin film. The sand particles should be in practical contact with each other throughout. The proportion of pore space in the standard sand used in these tests is essentially forty per cent. (It will be recalled that this sand is a clean, rounded river sand and represents an average grade and quality such as is obtainable along the streams of Iowa.) Theoretically, therefore, a volume of slaked lime equal to forty per cent of the total space enclosed by the sand is required to fill the open pores among the grains. If the lime could be confined to the pore spaces alone, still permitting the sand particles to touch at all possible points, such an amount of lime could be added without increasing the apparent volume of the sand, but this is not practically possible. As noted on an earlier page (page 10), white lime hydrates range in specific gravity from 2.12 to 2.32, and the magnesian averages 2.45. Assume an average for white limes of 2.22 and 2.65 for quartz sand. To be equal in volume to the voids in the sand there would be required in round numbers by weight thirty-six per cent of the dry lime hydrate. That is, with each sixty-four pounds of sand should be mixed thirty-six of slaked lime (estimated dry) to fill the space among the grains. There is required of the average dolomitic lime about thirty-nine in each hundred pounds of mixture to eliminate the voids. A liberal allowance would be forty per cent by weight in each case.

The results of the tests show the highest strength with 1:1 or fifty per cent mixture. As lower ratios of lime and sand were not employed, it is impossible to do other than speculate on the possible results from such mixtures. It seems probable that mixtures as low in lime as theoretically required to fill the voids may show higher strengths than the lowest proportion used in the foregoing tests. This limit of the series could profitably be extended to include even the neat lime so as to make the results conclusive. As the lime paste is ordinarily used in practice, it contains from fifty to sixty-five per cent of free moisture, the white limes carrying the larger amounts. In order to make calculations on the dry basis in mixing with sand, it is necessary to evaporate the water from a small sample of the paste, weighing before and after to determine its percentage. Practically, also, sands as they come from the bank contain a considerable percentage of fine material which decreases the voids. River sands range in the neighborhood of thirty-five per cent. The amount of voids can likewise be determined as directed in an earlier portion of this paper.

(d) The white limes require more water to slake properly, generate more heat in slaking, slake much more rapidly and reduce to a more uniform paste than the magnesian limes. The dolomitic limes set and harden more slowly but in many cases attain strengths so much greater than do the white limes as to be almost out of comparison. They will, therefore, stand greater dilutions of sand and still be sufficiently strong to meet the requirements of practical use.

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VOL. IV NO. II

Holding Power of Nails in Single Shear

By

M. I. Evinger.

JUNE 1908

IOWA STATE COLLEGE
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VOL. IV, NO. II. JUNE 1908

HOLDING POWER OF NAILS IN SINGLE SHEAR.

By

M. I. EVINGER

Published data on the holding power of nails when used with the ordinary kinds of building woods are found to be very meager. A few tests have been made showing the resistance of nails to direct pull but very little is known to have been done in investigating the holding power of the common wire nails when subjected to single shear. There is no doubt that data on the holding power of nails in single shear would be as valuable and interesting as that on the resistance of nails to direct pull.

In either case, it becomes evident after a few experiments and upon some thought that an attempt to find the exact tension or holding power withstood by a nail in being drawn from any particular kind of wood, would be wasted energy. This is due to the conditions of the case, i. e., the kind of wood, its state of seasoning, the coarseness of the grain, and the relation of the nail's surface to it, the method of the driving and of drawing, etc. These factors are variable and unforesoon. The results of the tests obtained herein are only approximate values of the actual condition and are valuable in that they are mostly comparative.

In the spring of 1907, Messrs. C. K. Morgan and F. Maresh, Seniors in the Iowa State College, Civil Engineering Department, carried out a series of experiments to obtain some data on the holding power of nails in single shear. As a rule nails are more often subjected to a shearing strain, for instance when two timbers are nailed together to prevent any sliding one upon the other. The results of their tests have been incorporated in this paper, and the writer here wishes to express his thaaks for the use of their data.

Five kinds of common building lumber were used in the ex-

Iowa State College of Agriculture & Mechanic Arts 8-14-19

periments; white pine, yellow pine, spruce, fir and oak. They were obtained from the stock piles in a local lumber yard and were not thoroughly seasoned at the time the experiments were started.

All the tests were made using the common wire nails. No tests were made using cut nails as they are almost off the market for ordinary use. The probable reason for the growth in popularity and predominance in the use of the wire nail seems to be due to the following facts. It is clean, looks well, and is regular in size; it is more easily driven and as a rule, not so liable to split the wood. Wire nails are some what cheaper, especially the smaller sizes. A few tests were made using the smal size barbed nails. The sizes of nails used are Nos. 8, 10, 12, 16, 20, 30, 40, 50 and 60 penny and were obtained partly at the local harware stores and also from the American Steel & Wire Company of Chicago.

The experiments were made in the Testing Materials Laboratory of the Iowa State College. An Olson 50,000 pound testing machine was used. The method used in making the tests was such as to obtain actual conditions as near as possible. So far as it has been able to learn from available data, other tests had been made simply by the pulling of two pieces of wood apart in the testing machine. A study of this method appeared to give too much of an element of tension on the nails tested due to eccentricity.

The figure on page 10 shows the method adopted for these tests. Three blocks A, B, and C of the same kind of wood, ranging from 12 inches to 15 inches in length were nailed together in the manner shown. The outer pieces A and C were of such a thickness as to correspond to the thickness used in actual practice for the particular size of nail tested. Any possible case of double shear was prevented by making the center piece B thick enough so that the nails after piercing the outer pieces, would not entirely penetrate it. The sizes of the pieces of lumber used were as follows: 1x4, 1½x4, 2x4, 3x4, 4x4, and 4x6 inches in cross section. The same number of nails were used on each side of the joint and when tested in compression as indicated by the arrows in the figure, a true shear was obtained and without tension or bending moment due to eccentricity. The ends of the blocks were carefully sawed off square in order to obtain an even bearing. The joint was then tested with the block B against the pulling head of the machine and with the blocks A and C resting upon the weighing table.

The loads were applied in increments and varied in amount according to the strength of the joint. Deflections for each increment added were obtained by measuring the vertical distance from the top of the weighing table to a knife edge which had previously been driven into the center block B. Care was exercised to avoid driving the spikes into knots or cracks.

The following table gives the average results of strength in pounds per nail of each size for the different woods used.

TABLE NO. I.

TESTS OF COMMON WIRE NAILS IN SINGLE SHEAR

Nails Used		Length of Nails in Inches	Gauge of Wire	Average strength in lbs. per nail for the total number of nails tested				
No. Penny	No. of Nails Tested			White Pine	Yellow Pine	Spruce	Fir	Oak
8	20	2½	10¼	230	294	187	478	656
10	20	3	9	235	324	237	475	608
12	8	3¼	9	318	625	332	360	665
16	20	3½	8	299	494	263	456	750
20	20	4	6	372	746	386	632	992
30	20	4½	5	540	1221	629	832	1392
40	20	5	4	656	1067	555	837	1580
50	12	5½	3	712	1425	563	985	1925
60	12	6	2	826	1783	794		2125
Barbed Wire Nails								
8	10	2½	10¼		425		307	
10	10	3	9		460		318	

The results have also been plotted in the form of curves and are shown at the end of this paper. The curves show plainly the relation between the ultimate shearing strength of the 2, 4, 6 and 8 nail joints of each size tested and for each kind of wood used.

The elastic limit of the woods was taken to correspond to that load at which the block B would return to its initial position when the load was removed. The elastic limit of all the woods, except oak, was found to correspond to a deflection of from 3-64 to 5-64 of an inch. The elastic limit of oak was, as a rule, not over 1-64 of an inch. During the tests it was observed that although the loads were applied steadily, the deflections per increment of load would vary. The joint would seem to stick for a moment and then give suddenly. This was probably due to the intermittent adjustment of the internal fibers of the wood to the different stresses.

The horizontal distances between curves on each set of curves represent the variation in ultimate strength per nail. The holding powers were obtained for each size of nail by dividing the total load per joint for each increment of load by the number of nails used in the joint.

The average strength per nail was found to vary up to 38 per cent; the average variation being about 6 per cent. Where the variations were noted, they were nearly always found to be due to the cracking of the joint while it was being tested. In a large number of the tests it was found that the maximum holding power of the joint per nail, of the same size, was almost identical whether two or eight nails were used.

The results seem to indicate that the strength of a joint would vary directly as the number of nails used.

The results shown in table No. 1 seem to indicate that the holding power varies directly as the size of the nail and as the depth of penetration.

Although an extended series of tests was not made, the following facts were noted in regard to seasoning of the woods used. The soft non-resinous woods showed a loss of 8 to 19 per cent due to seasoning while the hard woods containig resin developed from 20 to 32 per cent greater strength when seasoned than when green. The seasoning increased the tendency toward splitting. Oak showed about 30 per cent better qualities when seasoned than when green. Those timbers having a loose fibre structure have a lower resistence to shear on nails than those having compact fibre structures. Of the woods used in the tests, spruce, white pine, fir, yellow pine and oak, their degree of resistance to shear on the nail seems to be in the order named, oak having the greatest resistance.

A few tests were made with barbed nails with the following results. The barbed nails were about 40 per cent stronger in yellow pine than the smooth wire nails of the same size. The tests of the barbed nails were made with seasoned fir while the smooth nails were tested in green fir. This accounts for the barbed nails showing a lower corresponding value per nail.

It may be of interest as a matter of information and for the sake of comparison to incorporate Tables Nos. 1 and 2 from Engineering News, Vol. 31, on "Experiments to Determine the Holding Power of Nails and Drift Bolts," by Mr. F. W. Clay, C. E., at Cornell University in 1893.

These tables are shown at Tables Nos. II and III on page 8. The results in Table No. II, were obtained by driving the nails into hard red oak, and pulling them by means of an Olsen testing machine. From these results it is seen that the cut nail is superior to the wire nail, as far as direct tensile holding power is concerned.

The results in Table No. III were obtainted by nailing two short pieces of wood flat together so that their ends would not match, and subjecting them to a compressive stress in the direction of the joint by means of the Olsen machine. By this method the nails were subjected to a shearing strain. A better idea of the method may be more clearly obtained by thinking of the Block C in the figure on page 10 as removed and the Blocks A and B subjected to the forces as indicated by the arrows. As stated before this method seems to give an element of eccentricity but it is possible that the conclusions drawn upon the comparative results would be the same.

Table No. XIX from Bulletin No. 6, University of Illinois Engineering Experiment Station, on "Holding Power of Railroad Spikes," has also been incorporated herein and is shown as Table No. IV on page 9. Tests were made to determine the relation between the depth of penetration and the holding power of ordinary spikes and also screw spikes. The results show that the resistance varies directly with the depth of penetration. A spike is nothing more than a large nail, and the results obtained are much the same as for nails.

The common spikes used were 5½ and 6 inches in length, from 9-16 to 5-8 of an inch square and had comparatively smooth surfaces. The screw spikes ranged from 5 to 5½ inches in length; diameter of core 21-32 to 22-32 of an inch; projection of thread, 2-16 to 3-16 of an inch, and a pitch of 1 to 2. Holes for the spikes were bored 11-16 of an inch in diameter. As a result, a tight fit between the wood and spike was obtained, the thread cutting its own path.

The depth of insertion varied from 4½ to 5 inches. In Table No. IV the resistances given for the ordinary spikes are the results obtained where the spikes were driven to a uniform depth of 5 inches. The screw spikes, being of different lengths, necessarily were inserted to unequal depths. On account of the relation existing between the depth of penetration and the holding power, the resistances for the screw spikes are based upon a penetration of 5 inches.

TABLE NO. II.

RESISTANCE OF WIRE AND OF CUT NAILS TO A STRAIGHT PULL

Nail	Size, ins.	Area Sq. ins.	Weight grams	Stress in pounds	Ratio of stress to weight	Ratio of stress to area
Cut 12d.	4x0.21x0.185	3.16	19.5	1,990	10.1	620
Wire 20d.	4x0.186	2.34	15.1	1,220	81	530

Ratio, by areas, 6 to 5. Ratio, by weights, 5 to 4.

Cut 6d.	2.5x.12x.14	1.3	4.8	1,023	210	787
Wire 10d.	2.75x.136	1.17	5.0	579	116	482

Ratio, by areas, 1.4 to 1. Ratio, by weights, 1.8 to 1.

TABLE NO. III

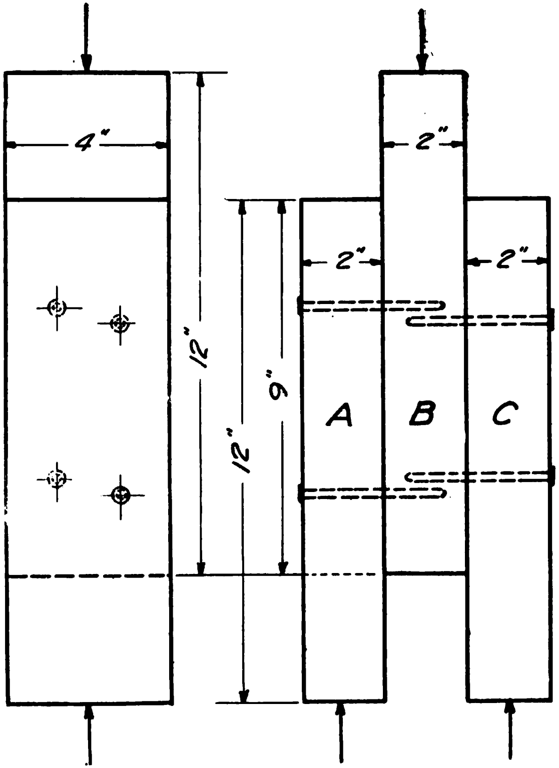
HOLDING POWER OF WIRE AND CUT NAILS SUBJECTED TO SHEARING STRAIN

Kind of wood, size, etc.	Size of nail	Area of nail in wood sq. in.	Weight of nail, grams	Stress per nail lbs.	Ratio of stress to weig't	Ratio of stress to area
Two slabs of 2-in. Georgia Pine	12d. cut	3.16	19.5	950	48.7	29.7
	20d. wire	2.34	15.1	745	49.3	31.8
Ditto, White Pine	12d. cut	3.16	19.5	905	46.4	28
	20d. wire	2.34	15.1	540	35.7	23
1-in. Ga. pine to 5-in. hemlock	20d. cut	hemlock 1.95	19.5	870		448
	20d. wire	1.6	15.1	647		404
1-in. Ga. pine to 3-in. oak	6d. cut	oak .73	4.8	527		110
	10d. wire	.71	5.0	526		105

TABLE NO. IV.

RELATIVE HOLDING POWER OF THE SCREW SPIKE AND OF THE ORDINARY SPIKE IN SEVERAL TIMBERS.

Kind of Tie	Kind of Spike	Resistance in Pounds for			Relative Resistances		
		1/8 In. Pull	1/4 In. Pull	Max. Resist	1/8 In. Pull	1/4 In. Pull	Max. Resist
Water Oak	Ordinary Screw	2870 4888	5730 9180	6780 12190	100 170	100 160	100 179
Black Oak	Ordinary Screw	2910 4760	5890 10420	7230 14110	100 164	100 177	100 203
Red Oak	Ordinary Screw	2950 4900	5350 10400	7730 13560	100 166	100 194	100 176
White Oak	Ordinary Screw	3510 6250	5950 11900	7870 12630	100 178	100 200	100 188
Ash	Ordinary Screw	3570 5700	5200 10470	7730 12760	100 162	100 200	100 165
Beech	Ordinary Screw	2600 6450	5490 13140	8840 16230	100 248	100 221	100 238
Elm	Ordinary Screw	2380 5120	5580 10090	7500 13690	100 215	100 181	100 183
Poplar	Ordinary Screw	2830 3880	5290 6210	5670 7490	100 137	100 117	100 132
Chestnut	Ordinary Screw	2850 3690	4070 6340	5200 8700	100 129	100 155	100 167
Sweet Gnm	Ordinary Screw	3230 5430	4120 7710	5300 8280	100 167	100 162	100 156
Loblolly Pine	Ordinary Screw	2920 5750	3500 9050	4300 10620	100 197	100 258	100 247



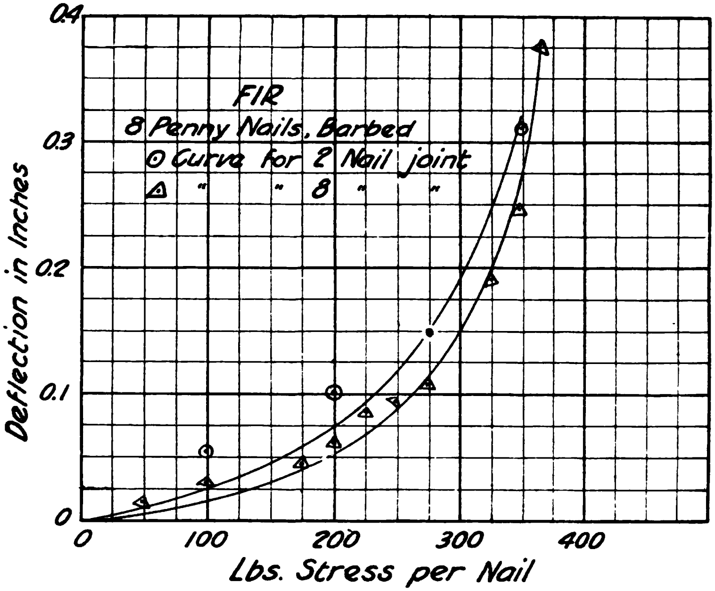
SINGLE SHEAR TESTS

Common Wire Nails

Showing the manner in which the joint was made and the way in which the forces were applied. The joint is a typical 4 nail joint when the 12 or 16 penny nails were tested.

Deflections were found by measuring the amount which the outside pieces slipped past the middle piece of the joint.

No. 1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⊙ Curve for 2 Nail joint
△ " " 8 " "
Deflection in Inches
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0.1
0.2
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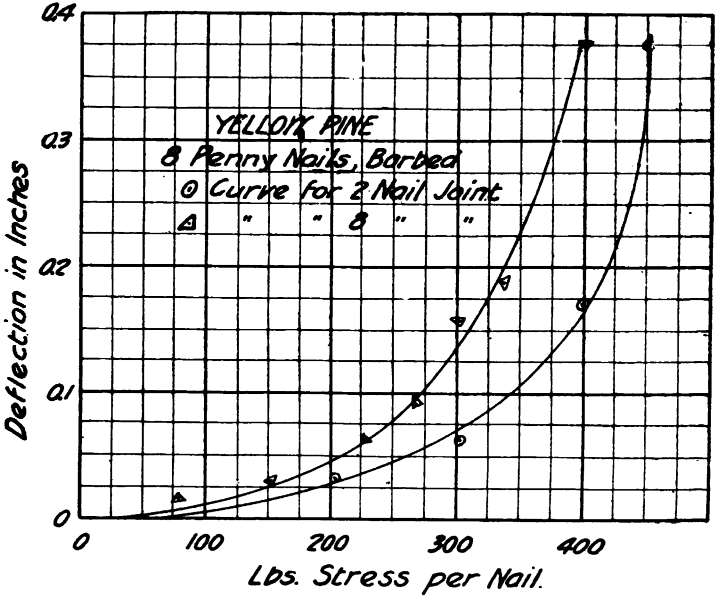
0.4
0
100
200
300
400
Lbs. Stress per Nail

No. 2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8 Penny Nails, Barbed
⊙ Curve for 2 Nail Joint
△ " " 8 " "
Deflection in Inches
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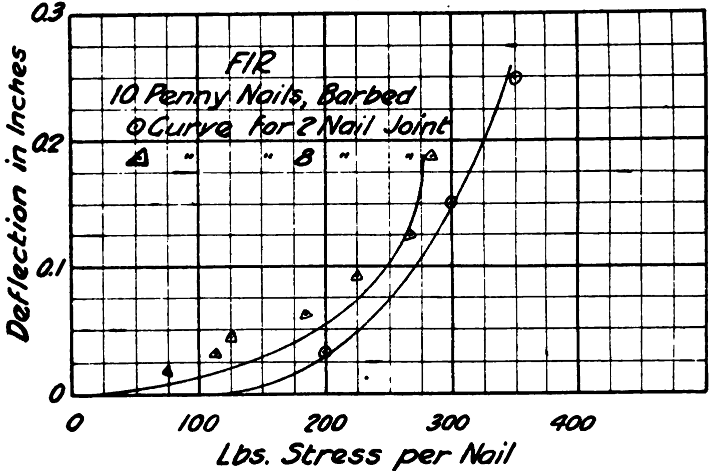
0.3
0.4
0
100
200
300
400
Lbs. Stress per Nail.

No.3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Deflection in Inches
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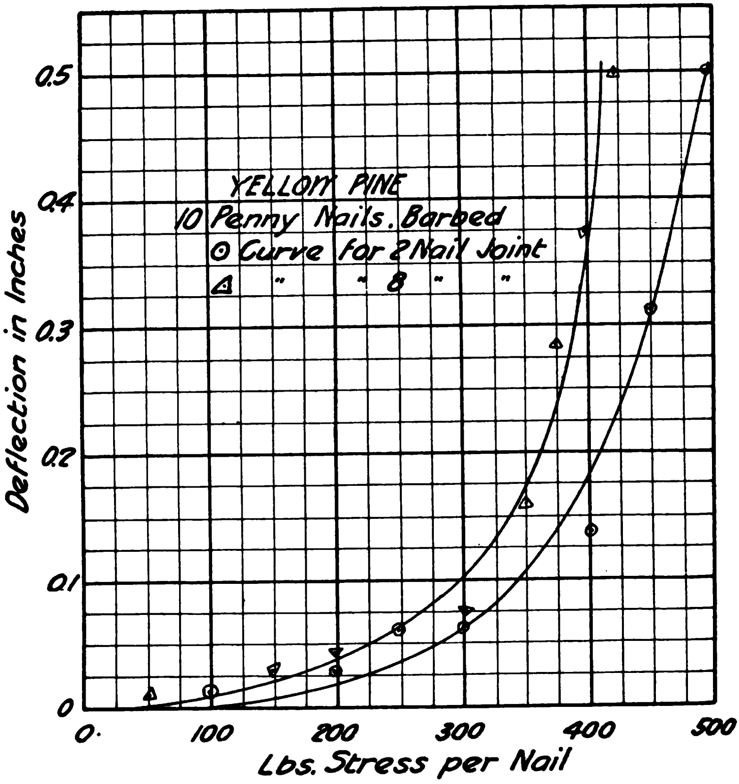
0.1
0
0
100
200
300
400
Lbs. Stress per Nail

No.4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Deflection in Inches
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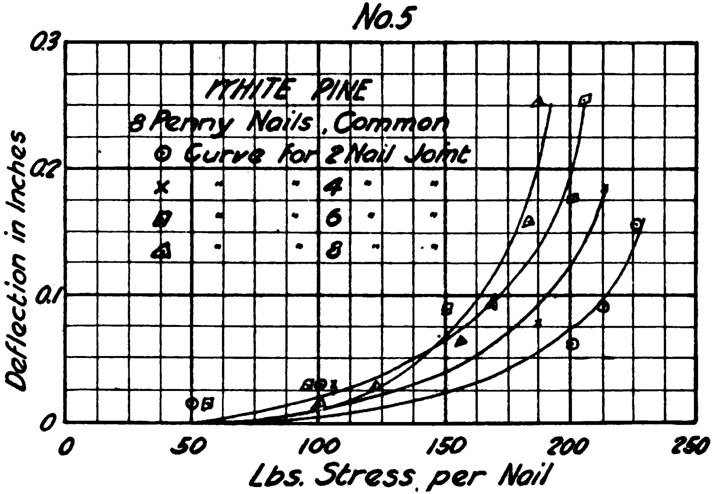
0
100
200
300
400
500
Lbs. Stress per Nail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WHITE PINE
8 Penny Nails, Common
⊙ Curve for 2 Nail Joint
× " " 4 " "
▢ " " 6 " "
△ " " 8 " "
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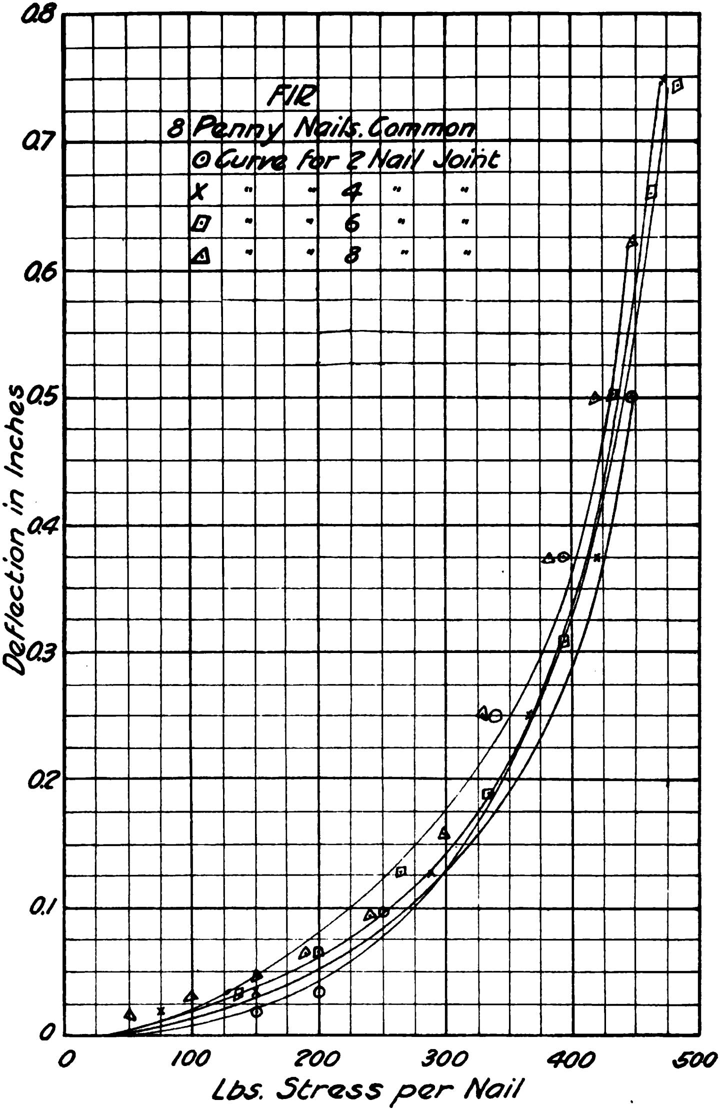
0.2
0.1
0
Deflection in Inches
0
50
100
150
200
250
Lbs. Stress per Nail

No. 6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⊙ Curve for 2 Nail Joint
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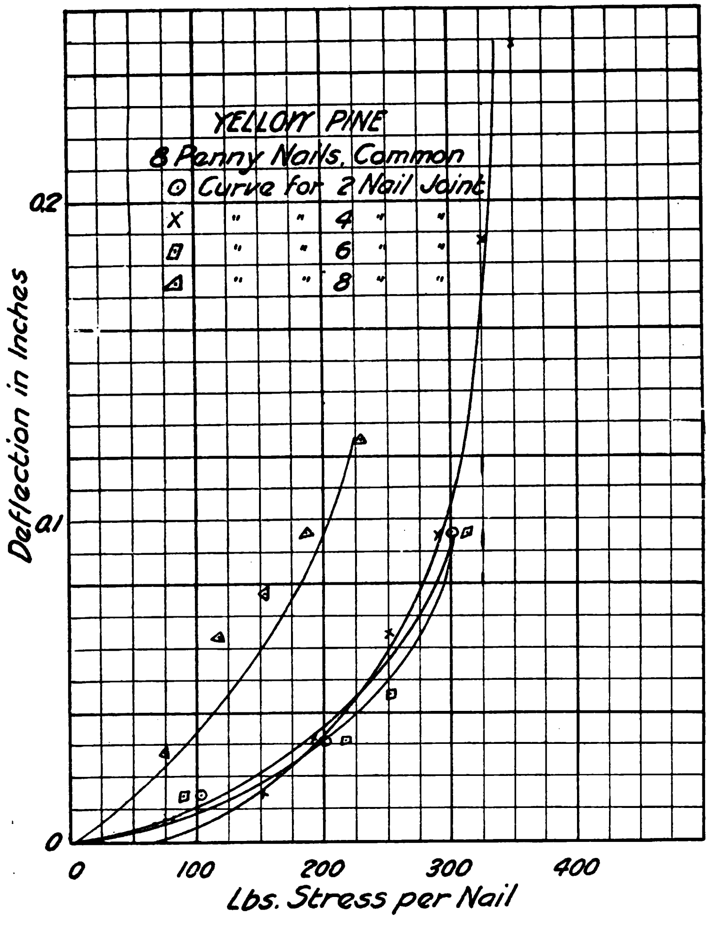
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
100
200
300
400
500
Lbs. Stress per Nail

No. 7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⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
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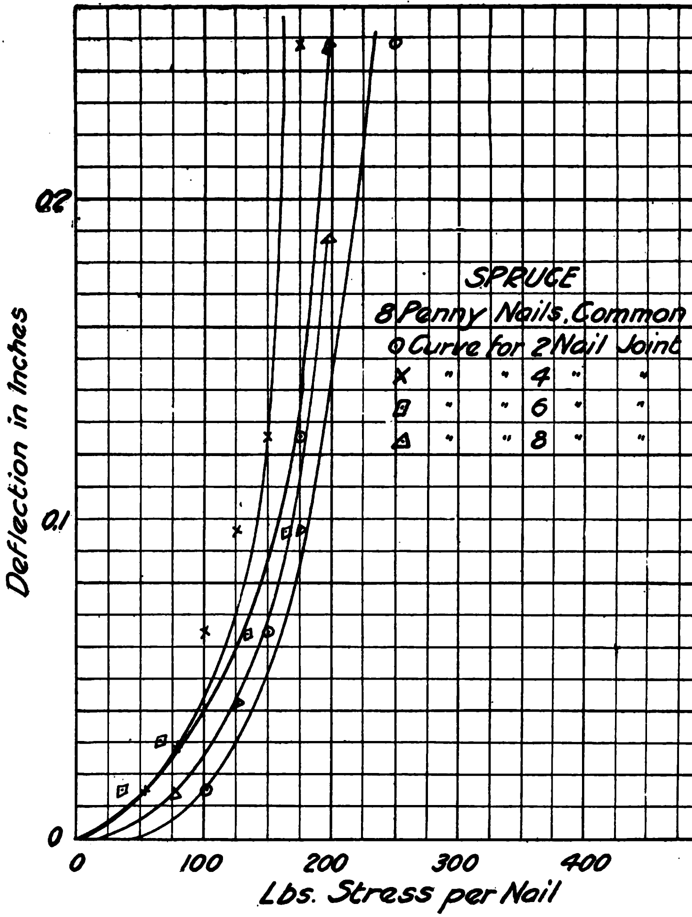
0.1
0
0
100
200
300
400
Lbs. Stress per Nail

No. 8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⊙ Curve for 2 Nail Joint
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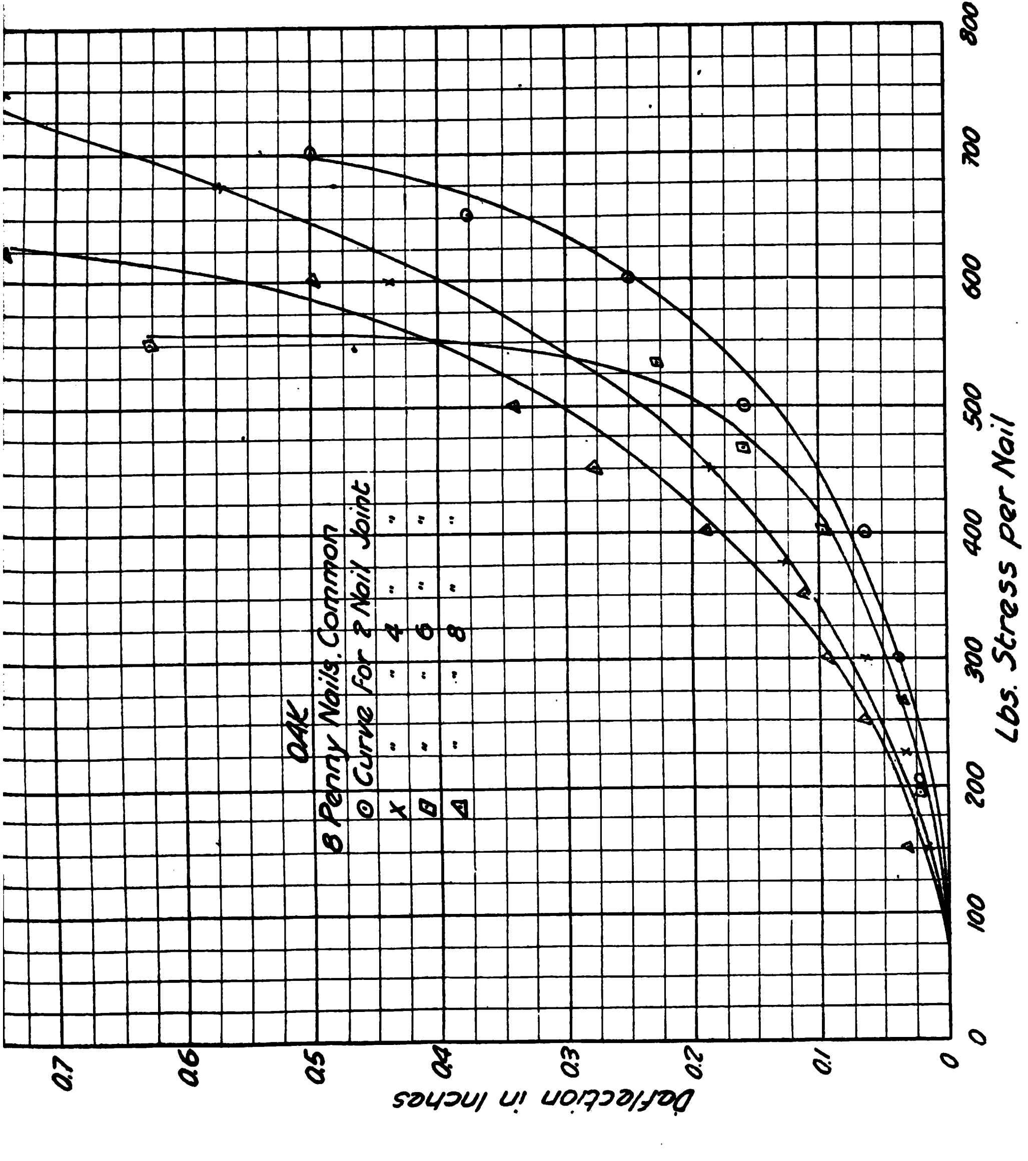
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.2
0.1
0
0
100
200
300
400
Lbs. Stress per Nail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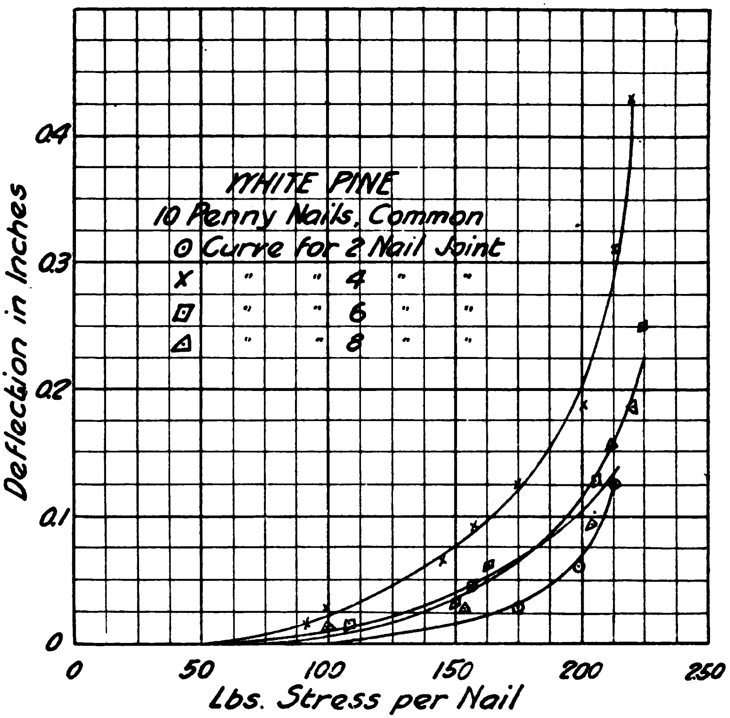
◻ " " 6 " "
△ " " 8 " "
Lbs. Stress per Nail
0
100
200
300
400
500
600
700
800
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

No.10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⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
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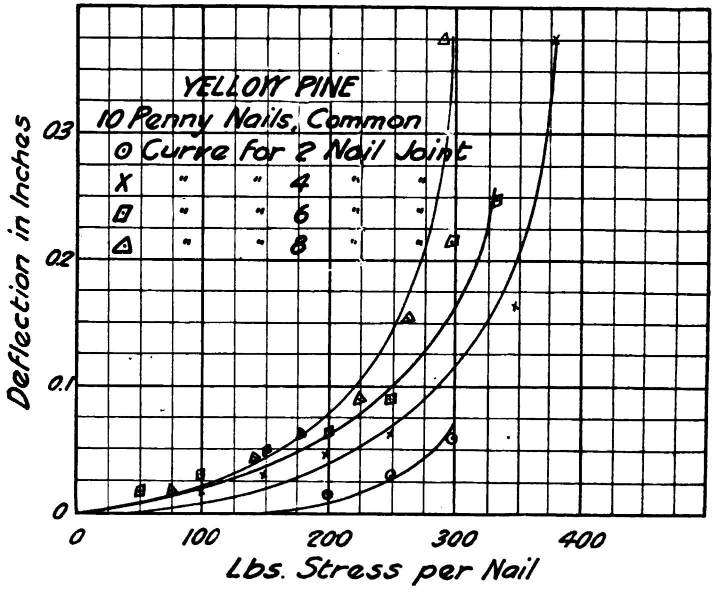
0.1
0.2
0.3
0.4
0
50
100
150
200
250
Lbs. Stress per Nail

No.11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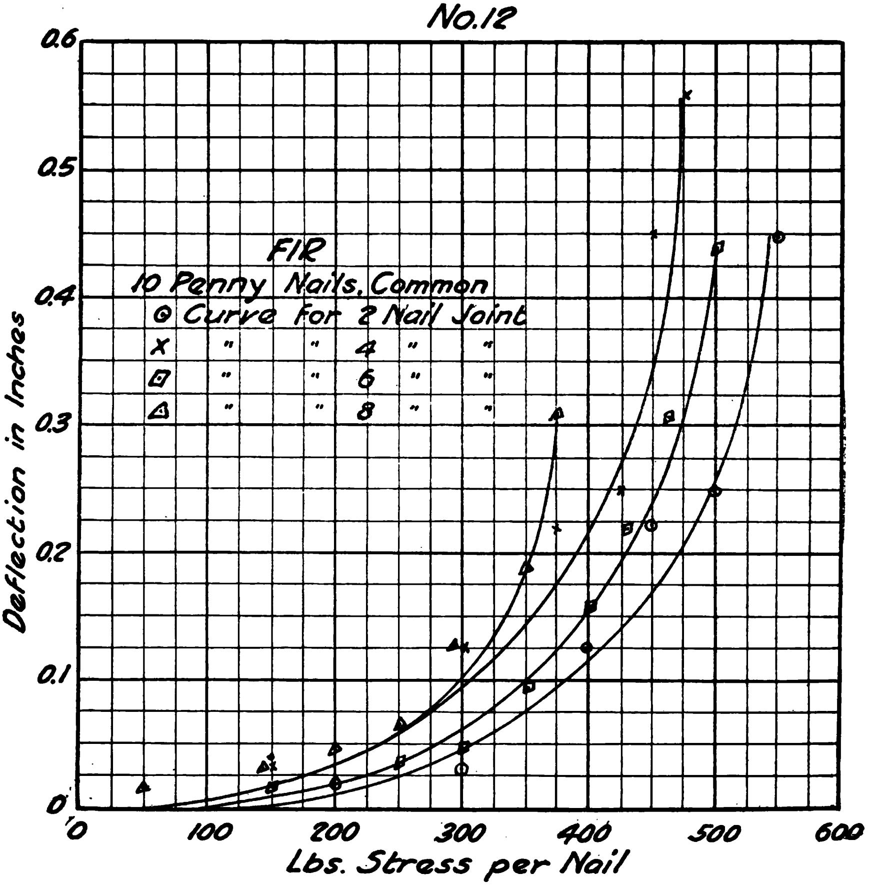
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0
100
200
300
400
Lbs. Stress per Nail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FIR
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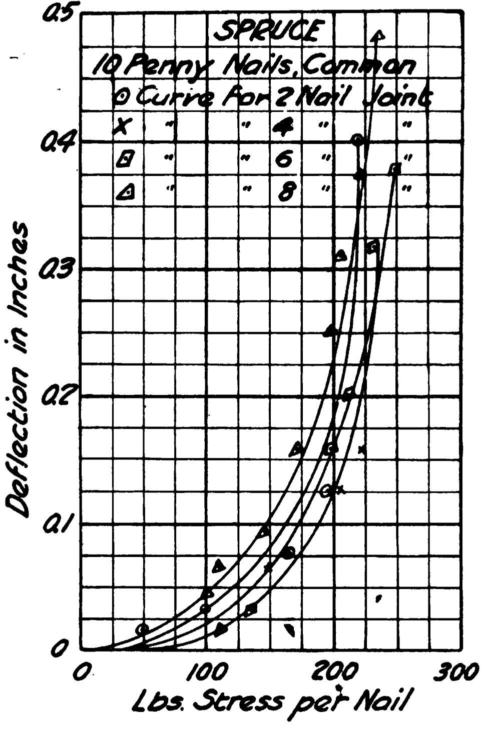
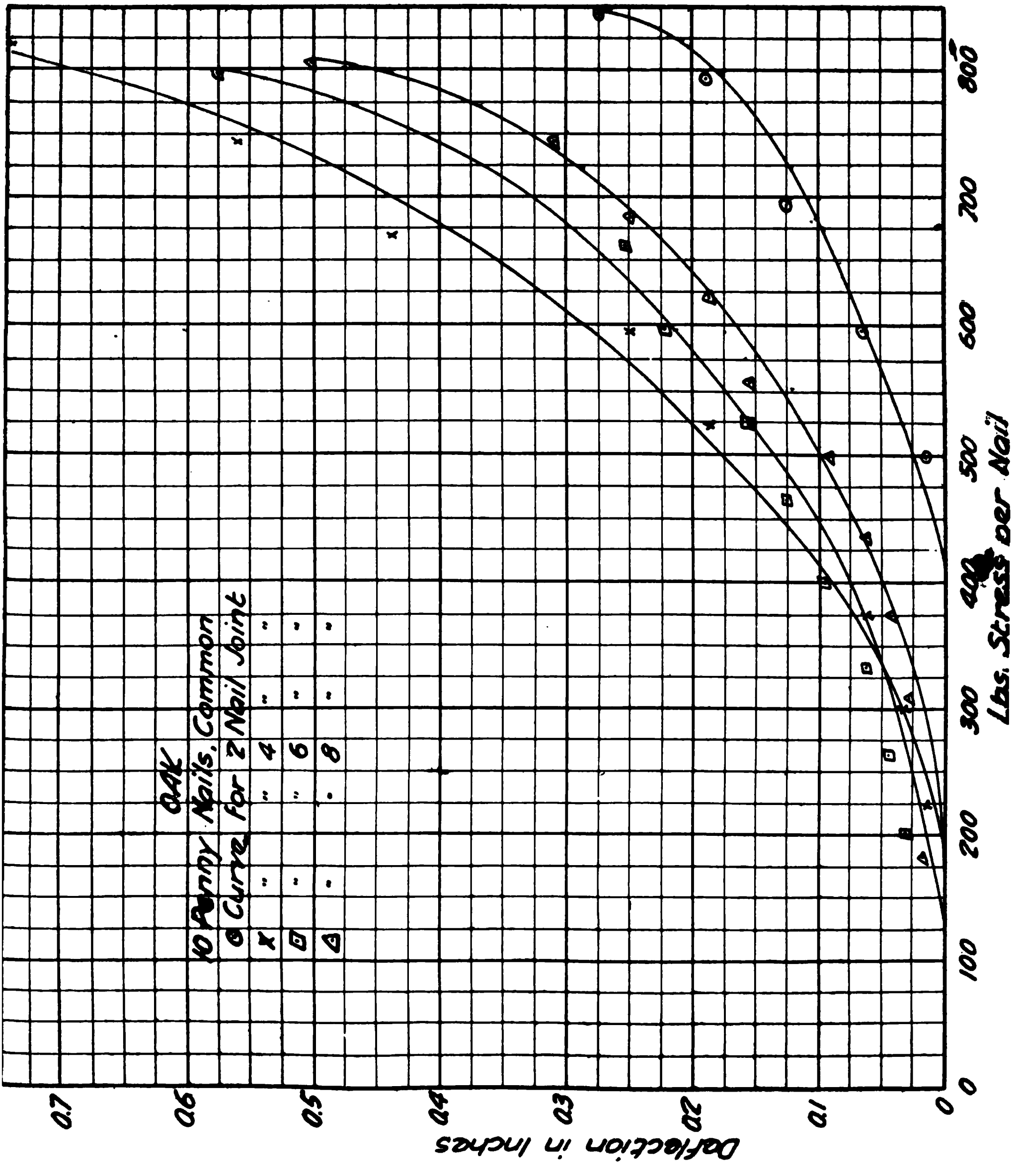
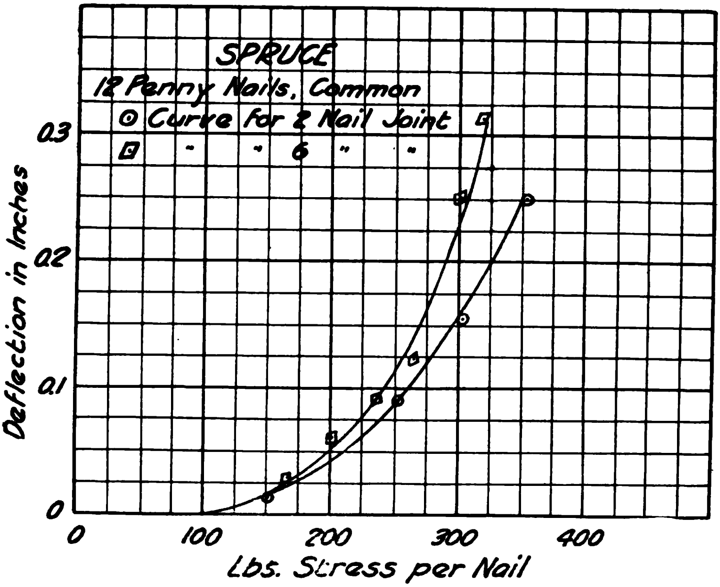
⊙ Curve for 2 Nail Joint
X " " 4 " "
▣ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
Lbs. Stress per Nail
0
100
200
300
400
500
600

No. 13

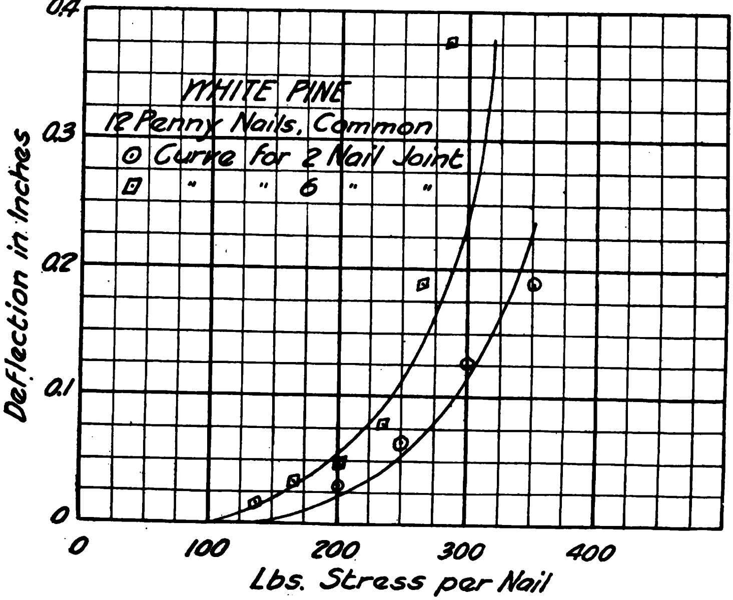

SPRUCE
10 Penny Nails, Common
⊙ Curve For 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
0.5
0.4
0.3
0.2
0.1
0
0
100
200
300
Deflection in Inches
Lbs. Stress per Nail


OAK
10 Penny Nails, Common
⊙ Curve for 2 Nail Joint
x " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
0
100
200
300
400
500
600
700
800
Lbs. Stress per Nail
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Deflection in inches

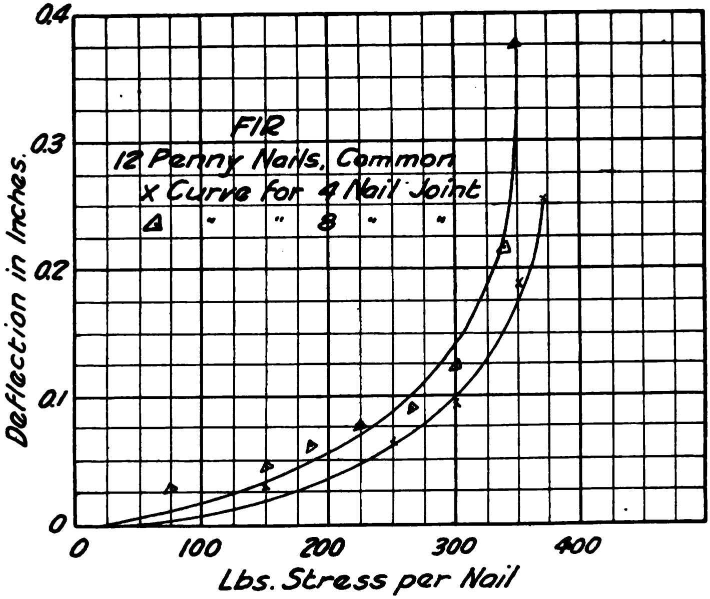
No. 15


SPRUCE
12 Penny Nails, Common
⊙ Curve for 2 Nail Joint
⊡ " " 6 " "
Deflection in Inches
0
0.1
0.2
0.3
0
100
200
300
400
Lbs. Stress per Nail

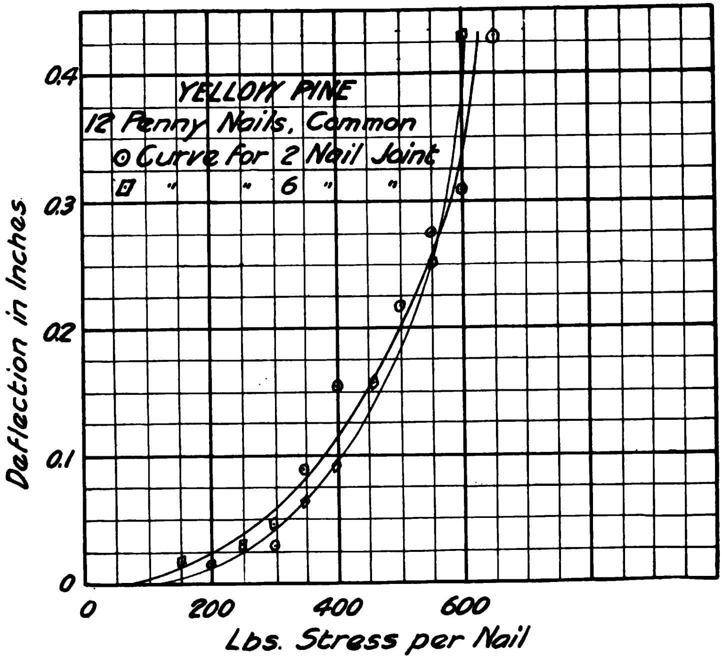
No. 16

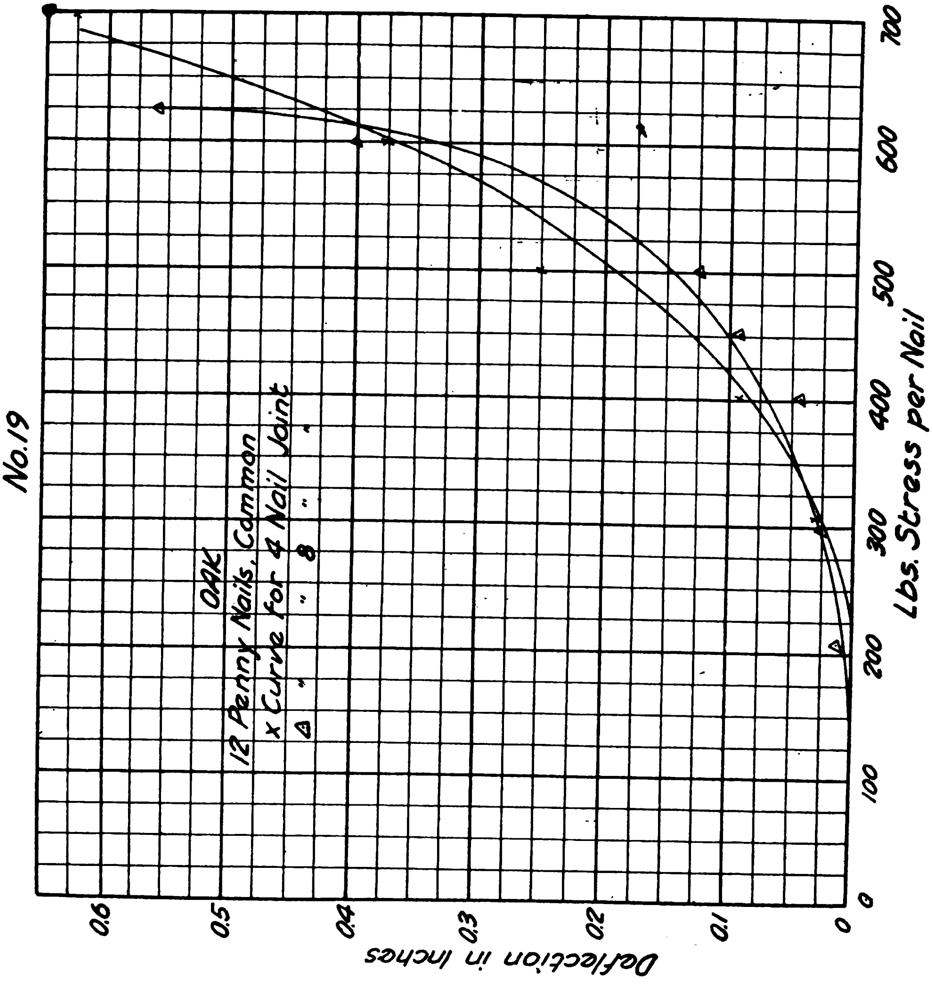

WHITE PINE
12 Penny Nails, Common
⊙ Curve for 2 Nail Joint
⊡ " " 6 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0
100
200
300
400
Lbs. Stress per Nail

No.17

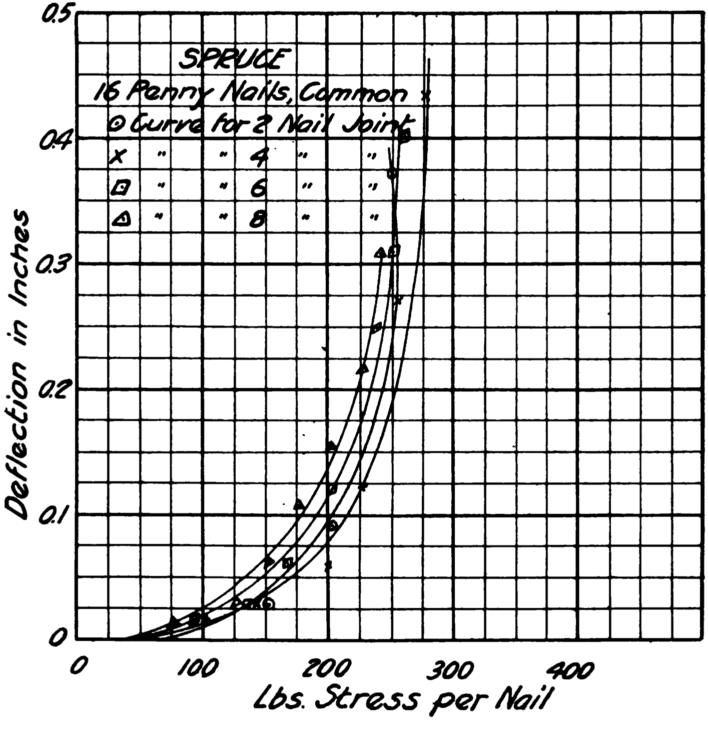

FIR
12 Penny Nails, Common
x Curve for 4 Nail Joint
Δ " " 8 " "
Deflection in Inches.
0.4
0.3
0.2
0.1
0
0
100
200
300
400
Lbs. Stress per Nail

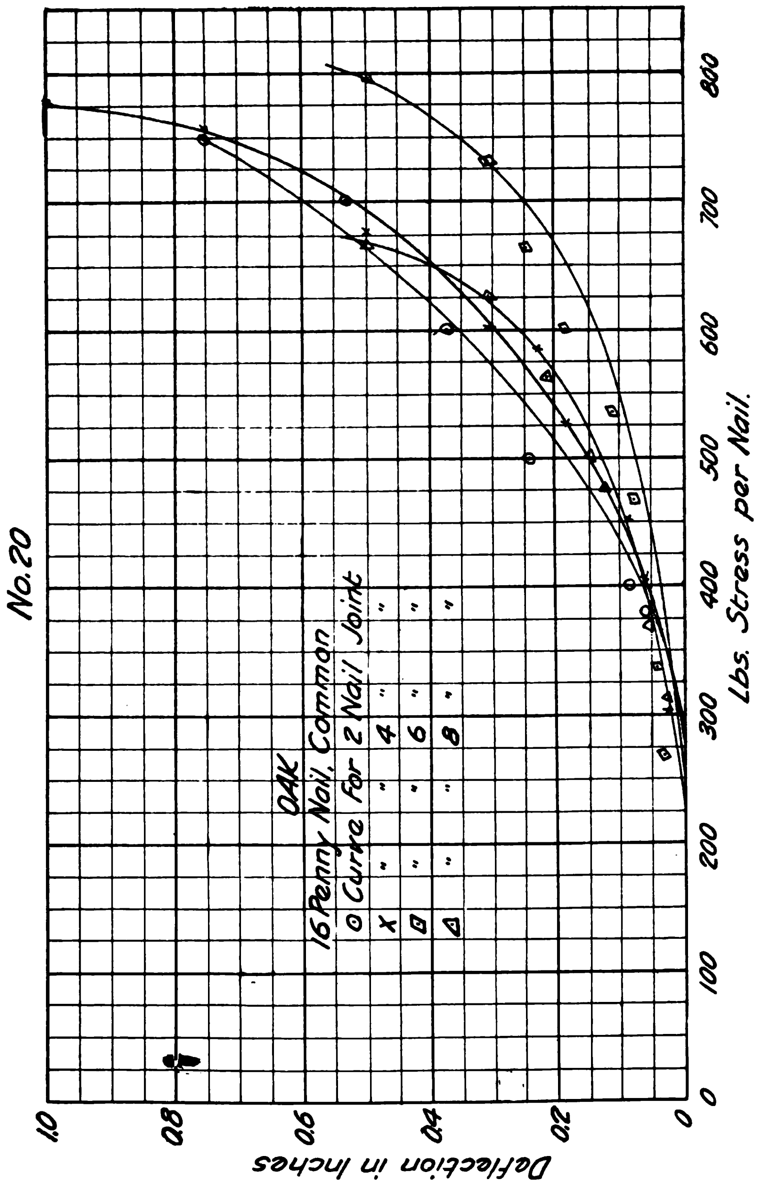
No.18


YELLOW PINE
12 Penny Nails, Common
⊙ Curve for 2 Nail Joint
⊡ " " 6 " "
Deflection in Inches
0.4
0.3
0.2
0.1
0
0
200
400
600
Lbs. Stress per Nail

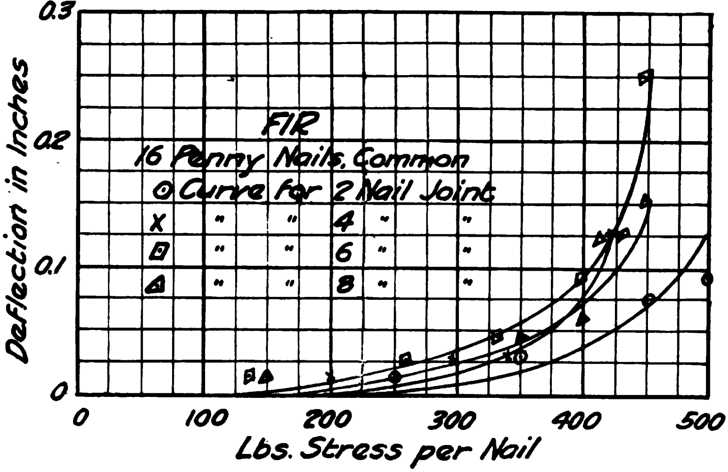

No.19
OAK
12 Penny Nails, Common
× Curve for 4 Nail Joint
Δ " " 8 " "
Lbs. Stress per Nail
0
100
200
300
400
500
600
700
Deflection in inches
0
0.1
0.2
0.3
0.4
0.5
0.6

No. 21

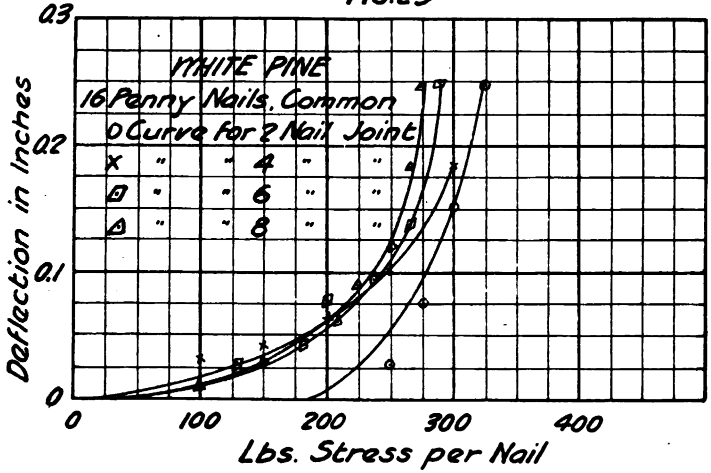

SPRUCE
16 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.5
0.4
0.3
0.2
0.1
0
0
100
200
300
400
Lbs. Stress per Nail

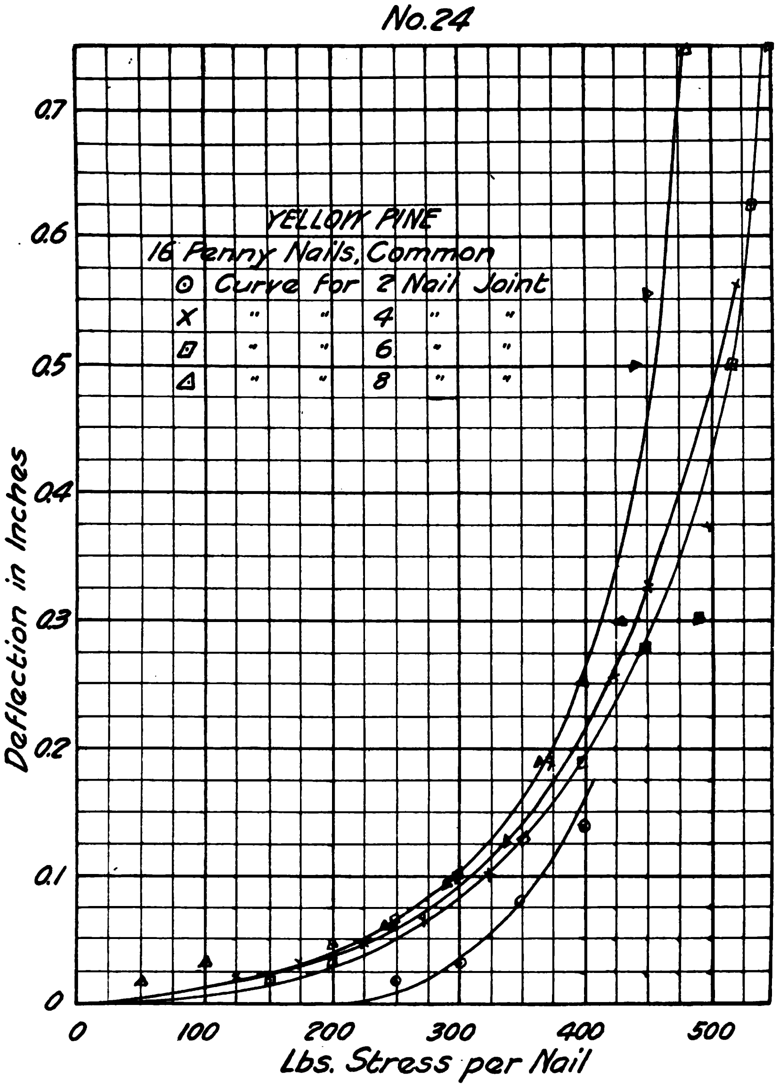

No. 20
OAK
16 Penny Nail, Common
⊙ Curve For 2 Nail Joint
X " " 4 " "
◻ " " 6 " "
△ " " 8 " "
Lbs. Stress per Nail.
0
100
200
300
400
500
600
700
800
Deflection in Inches
0
0.2
0.4
0.6
0.8
1.0

No.22

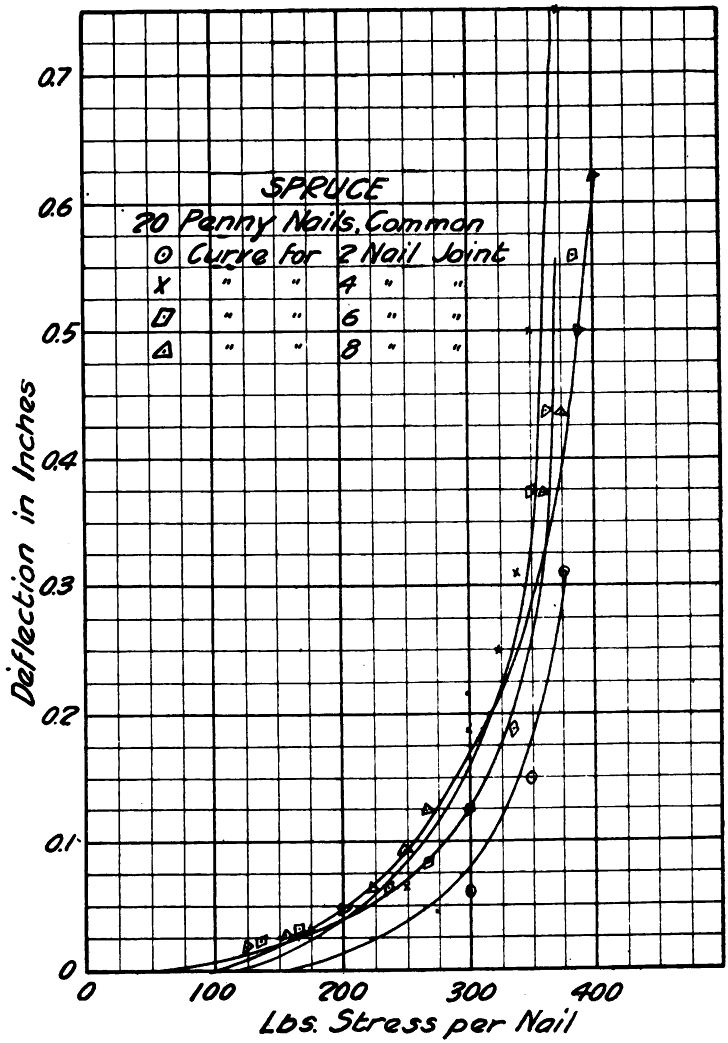

FIR
16 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
◬ " " 8 " "
Deflection in Inches
0.3
0.2
0.1
0
0
100
200
300
400
500
Lbs. Stress per Nail

No.23

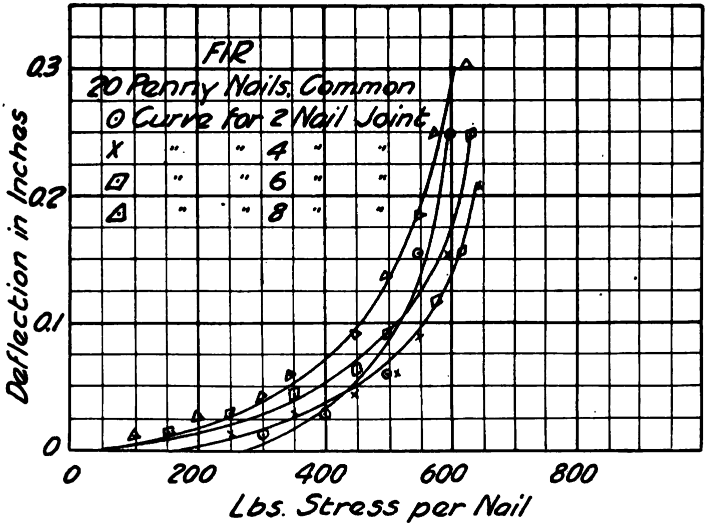

WHITE PINE
16 Penny Nails, Common
o Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
◬ " " 8 " "
Deflection in Inches
0.3
0.2
0.1
0
0
100
200
300
400
Lbs. Stress per Nail


No. 24
YELLOW PINE
16 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0
100
200
300
400
500
Lbs. Stress per Nail

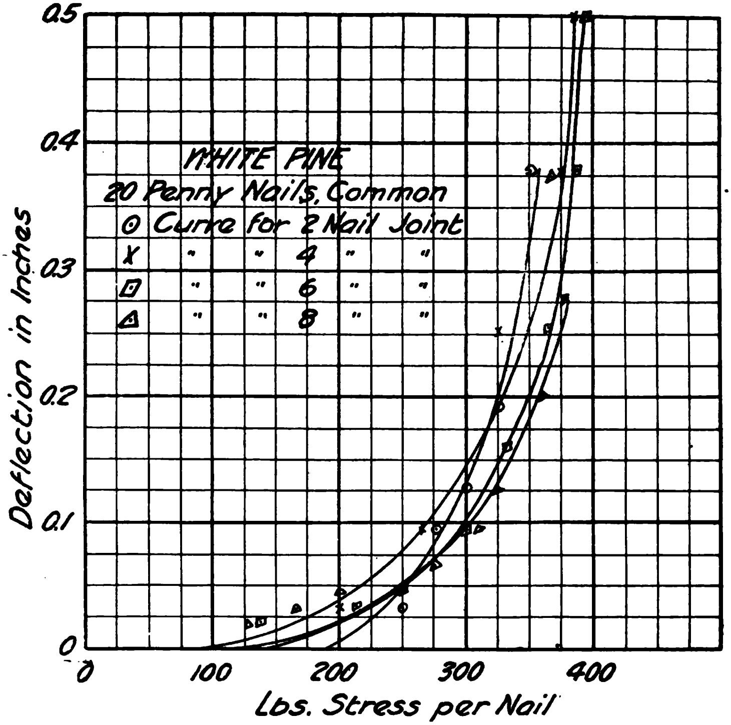
No.25


SPRUCE
20 Penny Nails, Common
○ Curve for 2 Nail Joint
X " " 4 " "
□ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0
100
200
300
400
Lbs. Stress per Nail

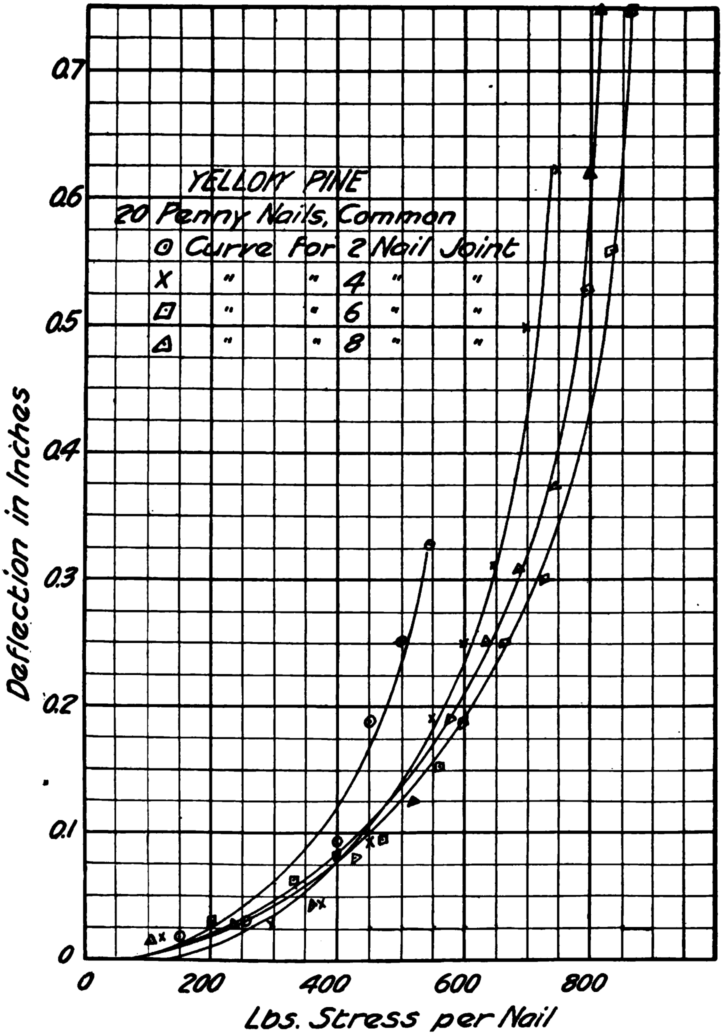
No. 26


FIR
20 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
▣ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0
200
400
600
800
Lbs. Stress per Nail

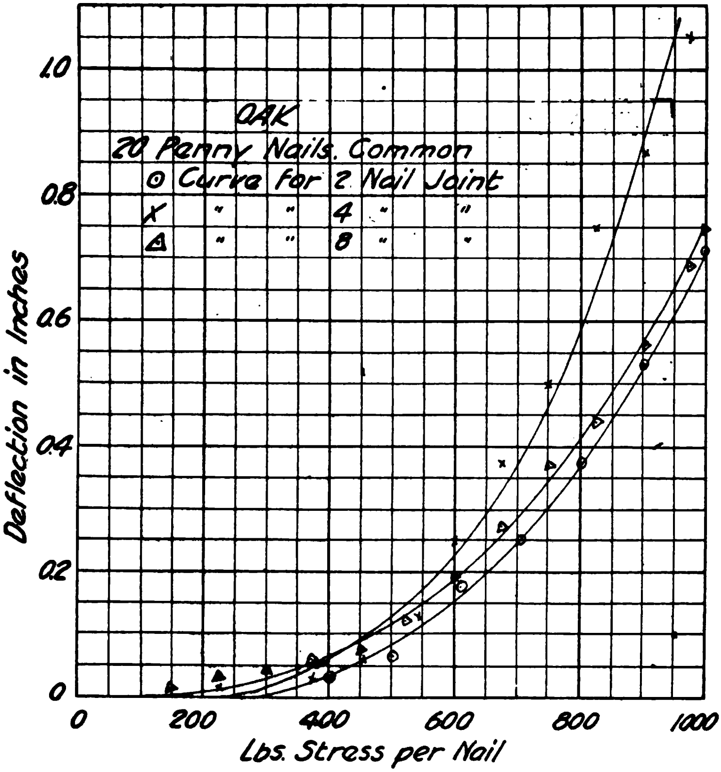
No. 27

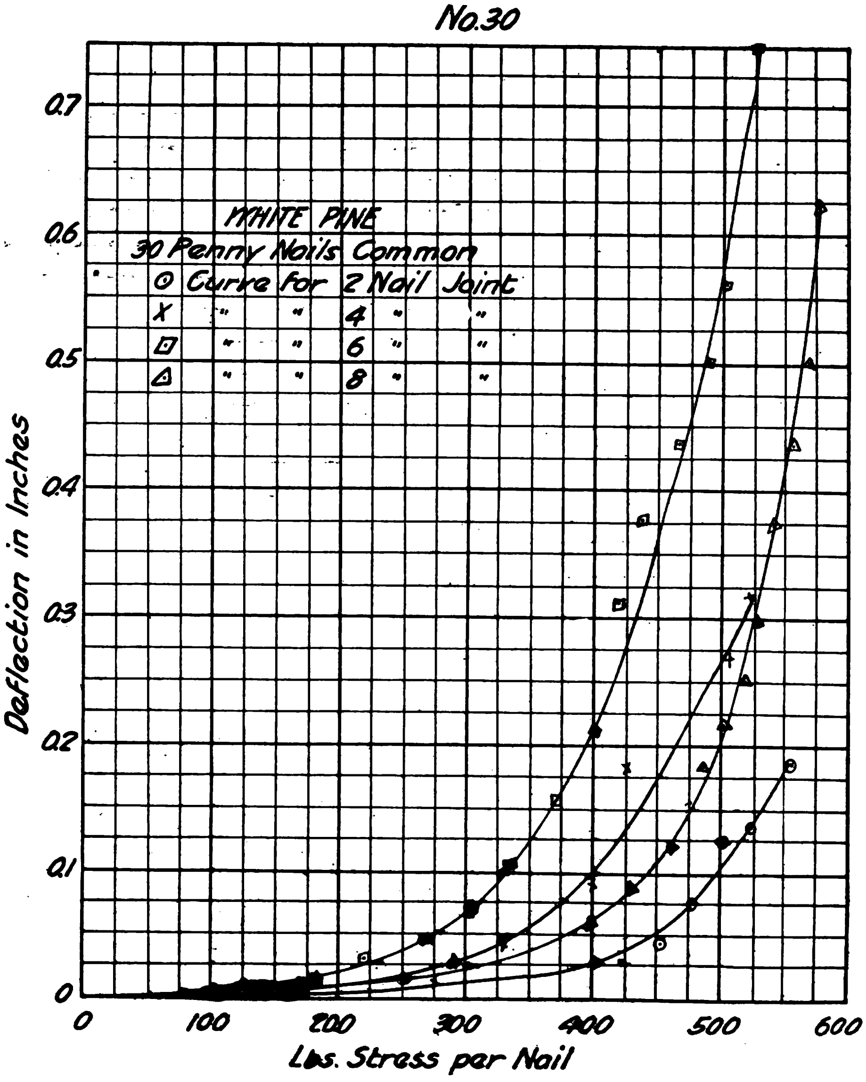

WHITE PINE
20 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
▣ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0
100
200
300
400
Lbs. Stress per Nail

No. 28

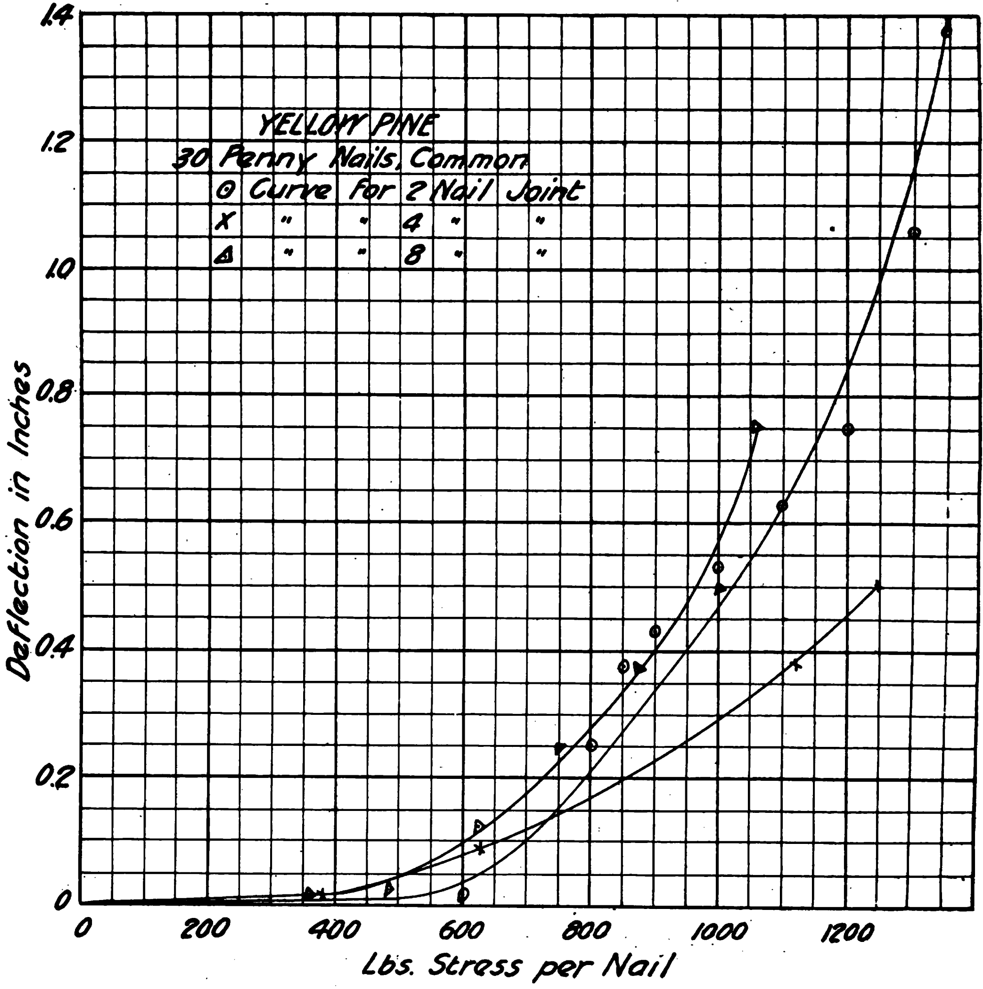

YELLOW PINE
20 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Deflection in Inches
0
200
400
600
800
Lbs. Stress per Nail

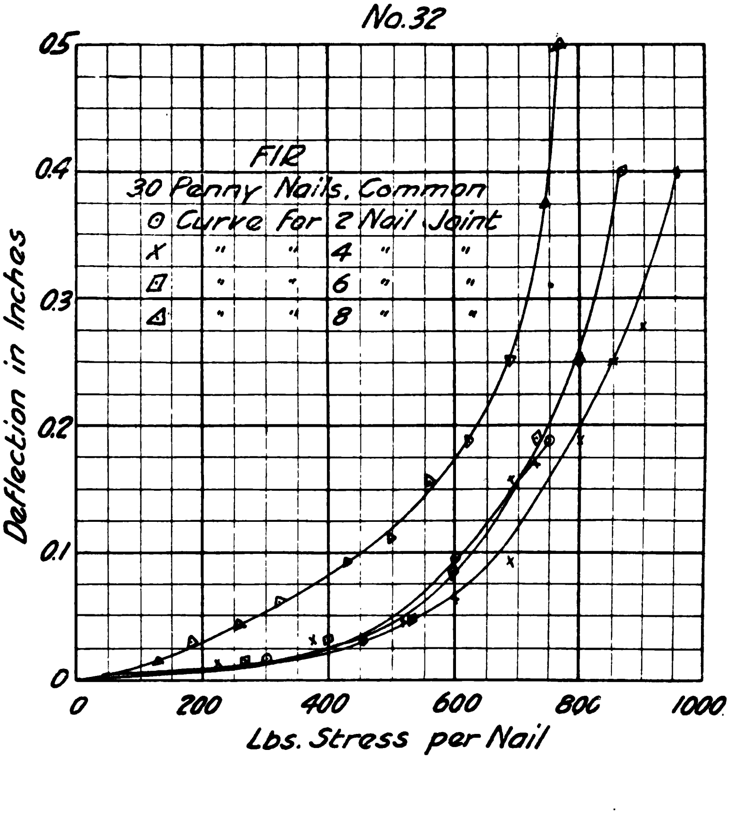
No. 29

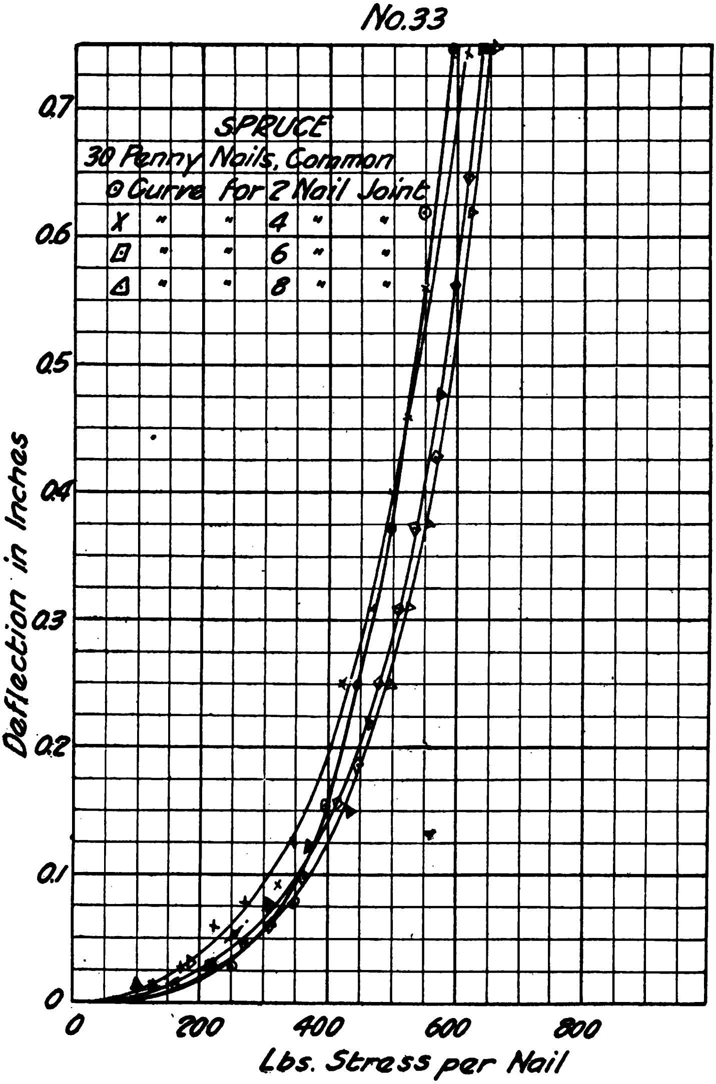

OAK
20 Penny Nails. Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
Δ " " 8 " "
Deflection in Inches
1.0
0.8
0.6
0.4
0.2
0
0
200
400
600
800
1000
Lbs. Stress per Nail


No.30
WHITE PINE
30 Penny Nails Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0
100
200
300
400
500
600
Lbs. Stress per Nail

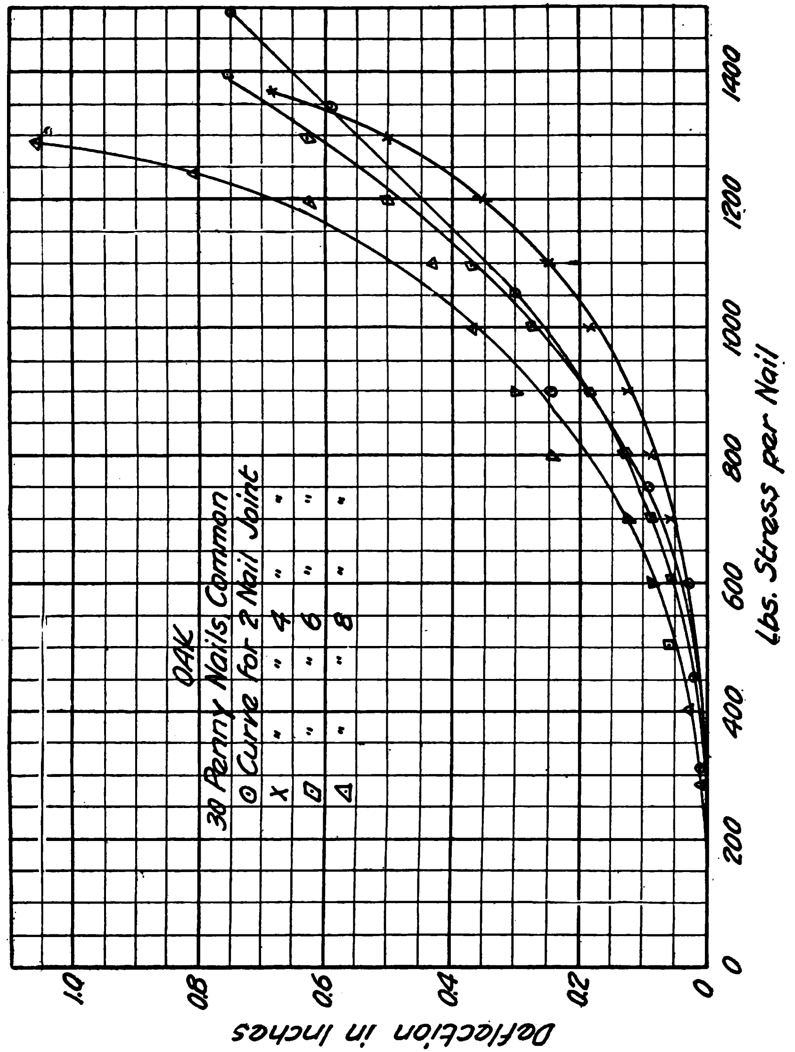
No.31

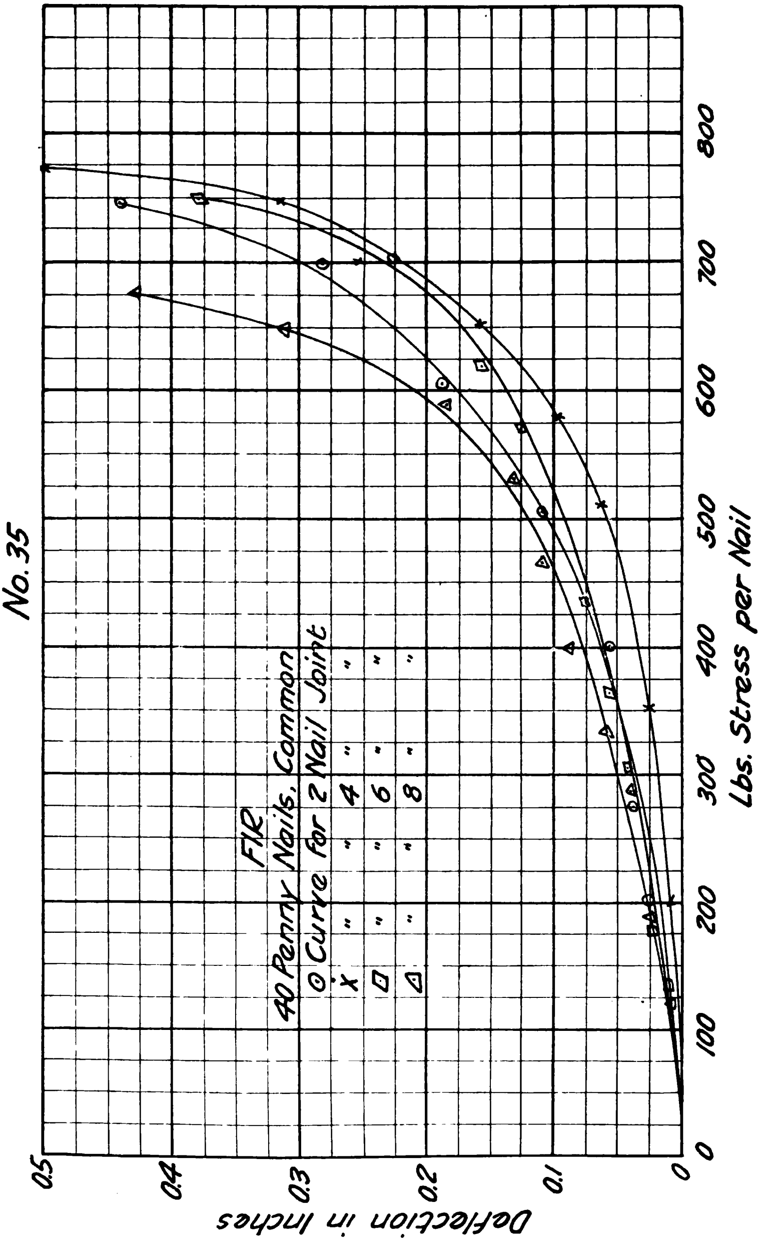

YELLOW PINE
30 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
Δ " " 8 " "
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
Deflection in Inches
0
200
400
600
800
1000
1200
Lbs. Stress per Nail


No. 32
FIR
30 Penny Nails, Common
⊙ Curve For 2 Nail Joint
X " " 4 " "
▣ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.5
0.4
0.3
0.2
0.1
0
0
200
400
600
800
1000
Lbs. Stress per Nail


No.33
SPRUCE
30 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
200
400
600
800
Deflection in Inches
Lbs. Stress per Nail

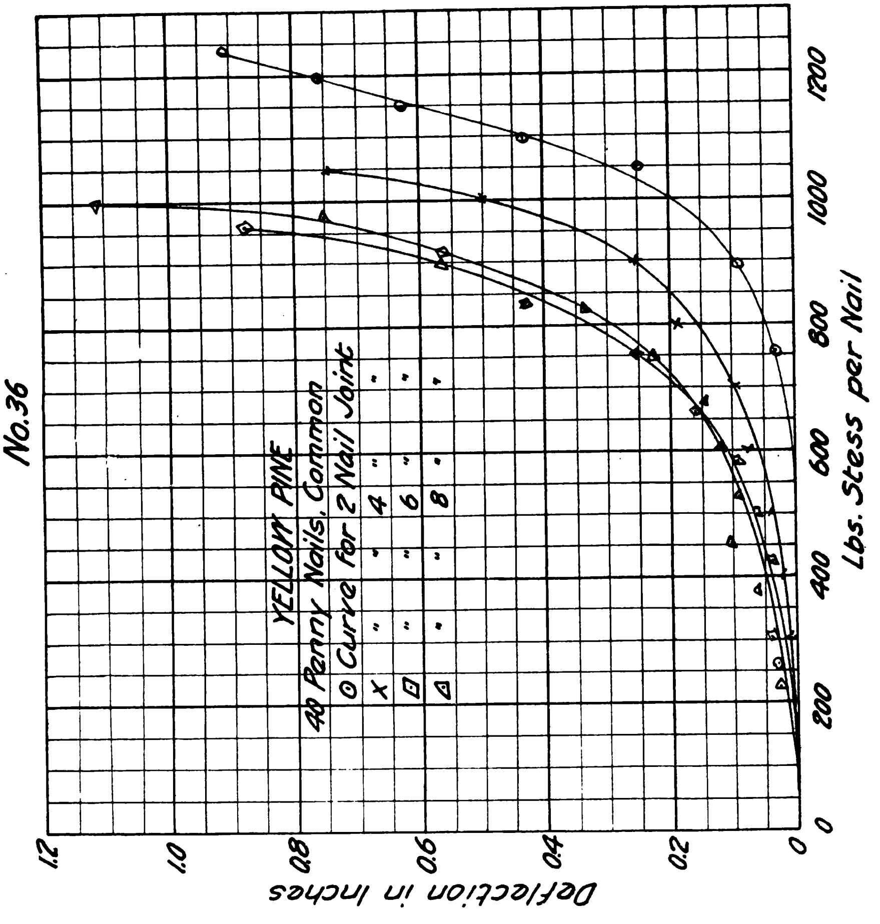
No. 34


OAK
30 Penny Nails Common
⊙ Curve for 2 Nail Joint
× " " 4 " "
◻ " " 6 " "
△ " " 8 " "
Lbs. Stress per Nail
Deflection in Inches
0
200
400
600
800
1000
1200
1400
0
0.2
0.4
0.6
0.8
1.0

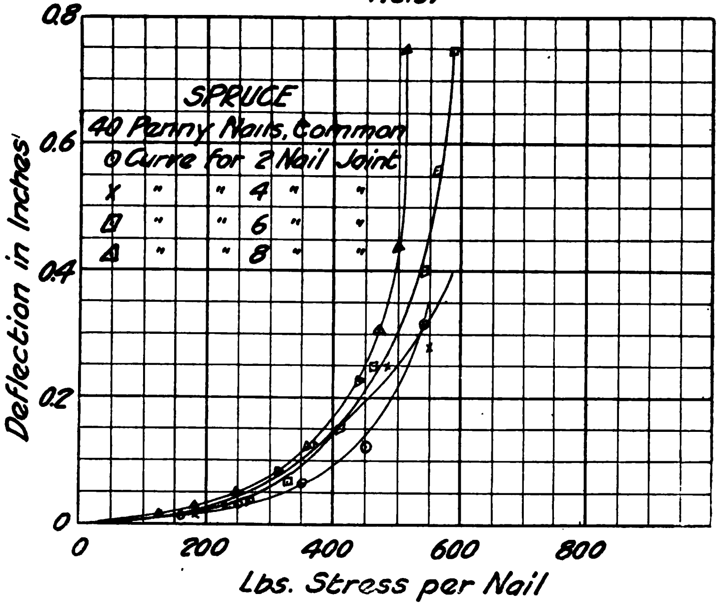


No. 35

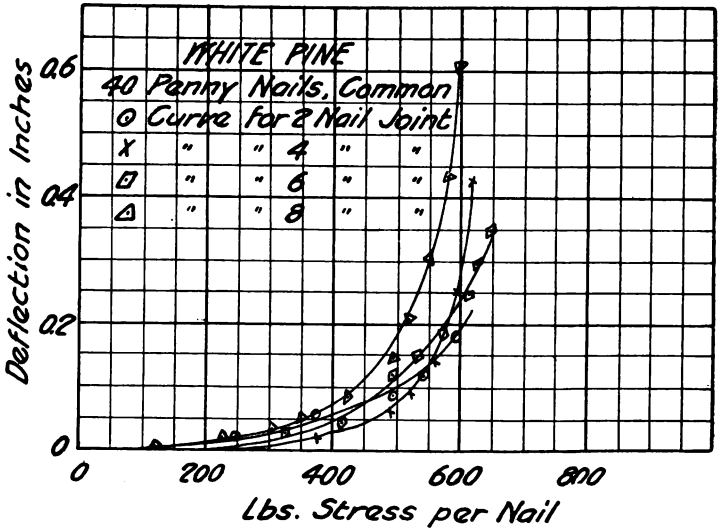
FIR
40 Penny Nails, Common
⊙ Curve for 2 Nail Joint
x " " 4 " "
□ " " 6 " "
△ " " 8 " "
Lbs. Stress per Nail
0
100
200
300
400
500
600
700
800
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5

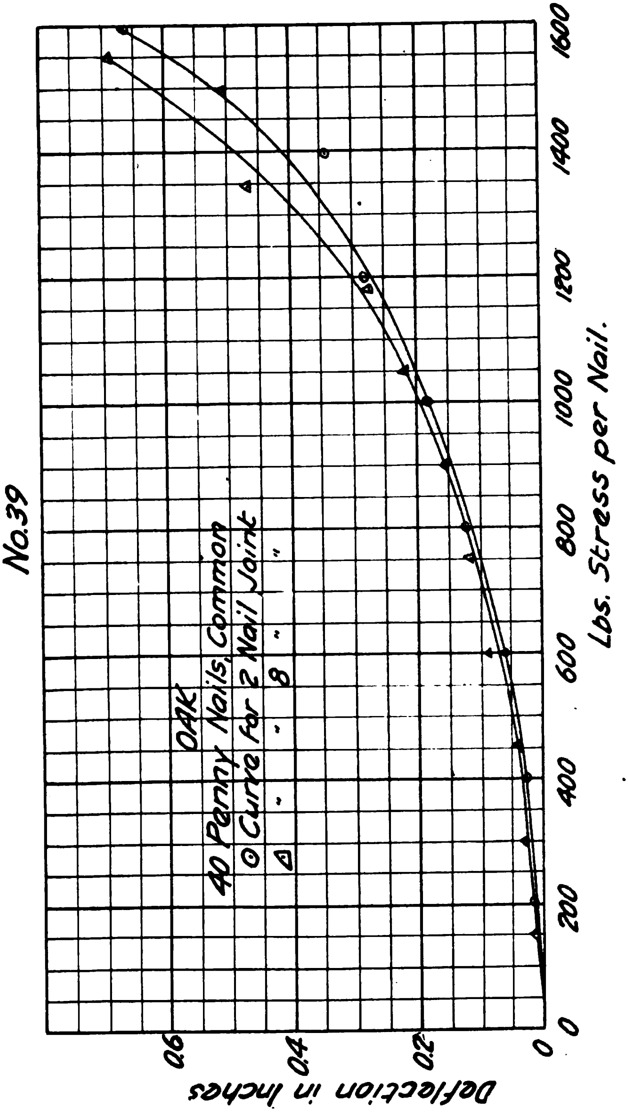

No.36
YELLOW PINE
40 Penny Nails, Common
⊙ Curve for 2 Nail Joint
× " " 4 " "
◇ " " 6 " "
△ " " 8 " "
Lbs. Stess per Nail
0
200
400
600
800
1000
1200
Deflection in Inches
0
0.2
0.4
0.6
0.8
1.0
1.2

No.37

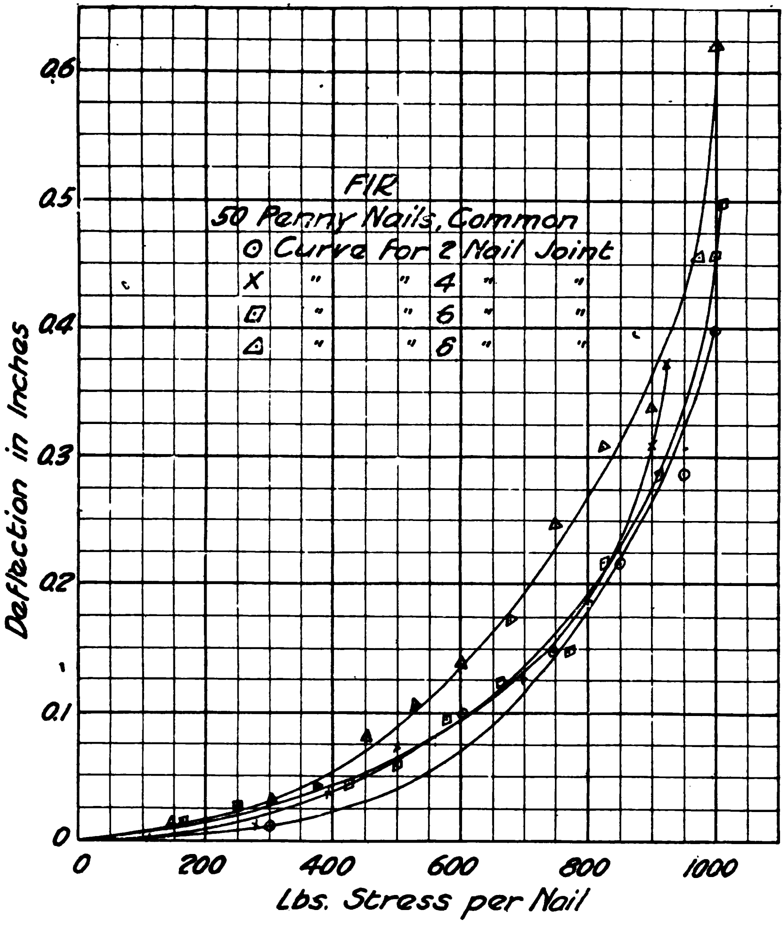

SPRUCE
40 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
▣ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.8
0.6
0.4
0.2
0
0
200
400
600
800
Lbs. Stress per Nail

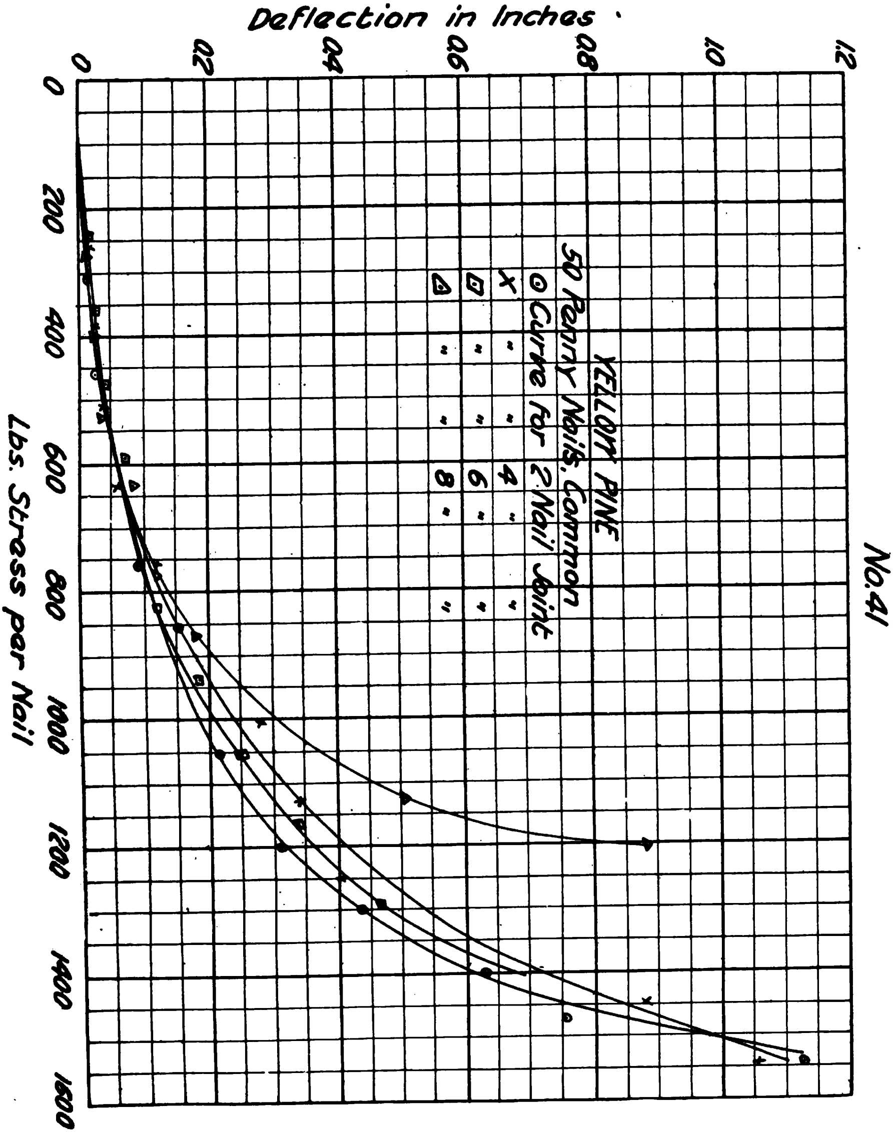
No.38


WHITE PINE
40 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
▣ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.6
0.4
0.2
0
0
200
400
600
800
Lbs. Stress per Nail

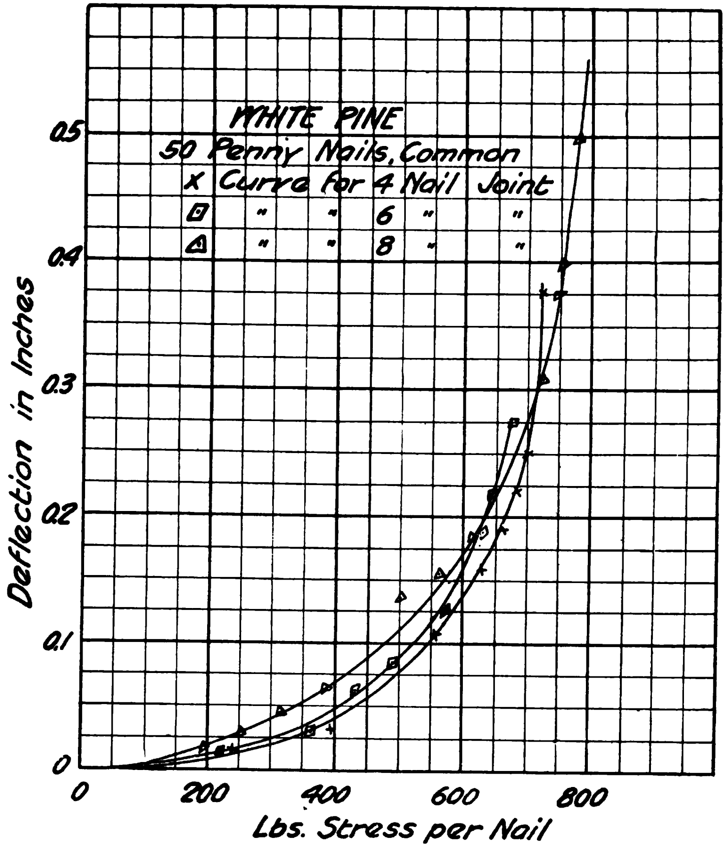

No.39
OAK
40 Penny Nails, Common
⊙ Curve for 2 Nail Joint
△ " " 8 " "
0
200
400
600
800
1000
1200
1400
1600
Lbs. Stress per Nail.
0
0.2
0.4
0.6
Deflection in Inches

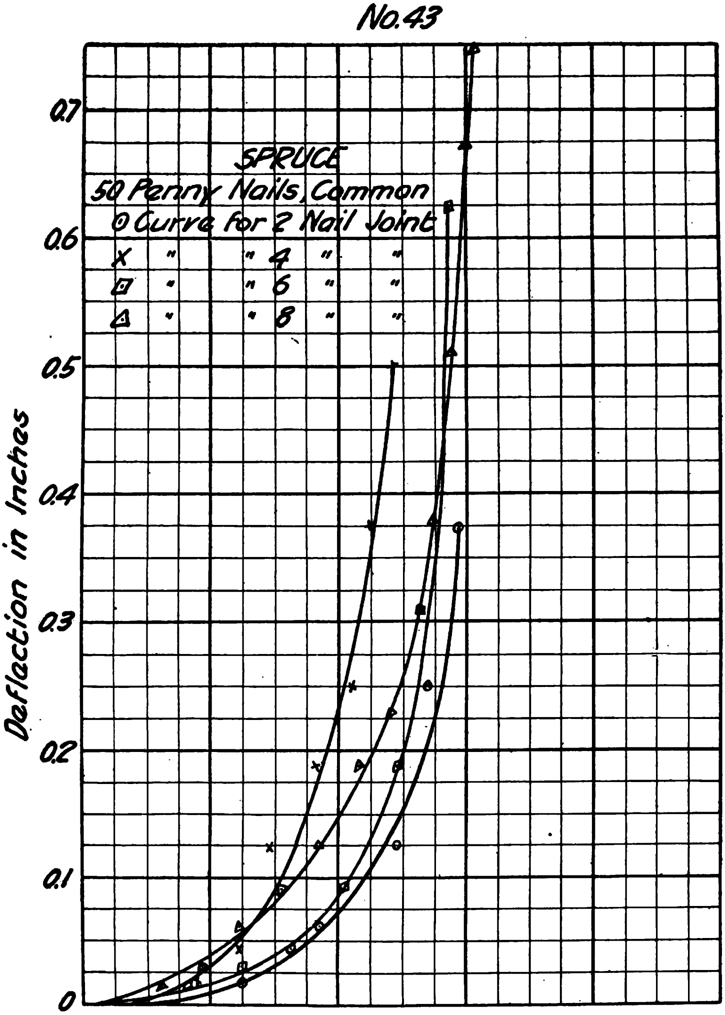
No. 40


FIR
50 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
0
200
400
600
800
1000
Lbs. Stress per Nail

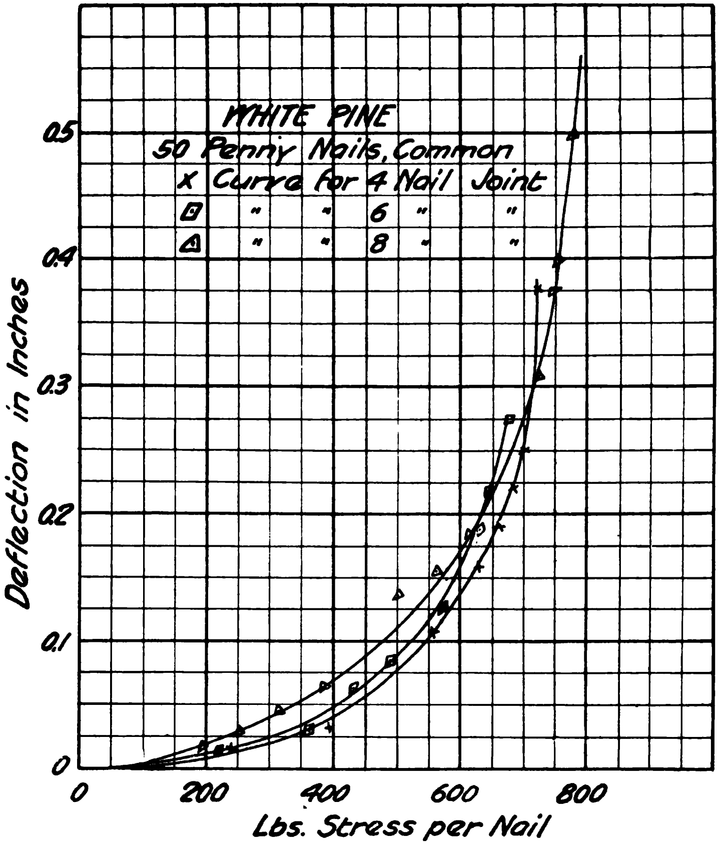

No.41
YELLOW PINE
50 Penny Nails, Common
⊙ Curve for 2 Nail Joint
× " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.2
0.4
0.6
0.8
1.0
1.2
Lbs. Stress per Nail
0
200
400
600
800
1000
1200
1400
1600

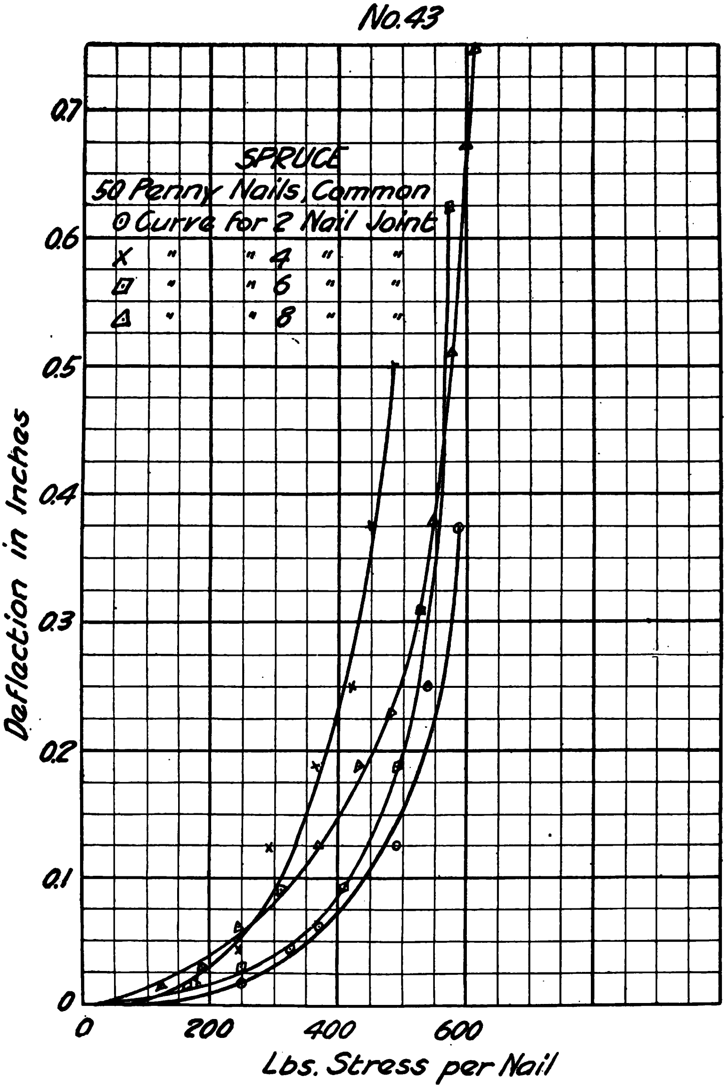
No. 42.

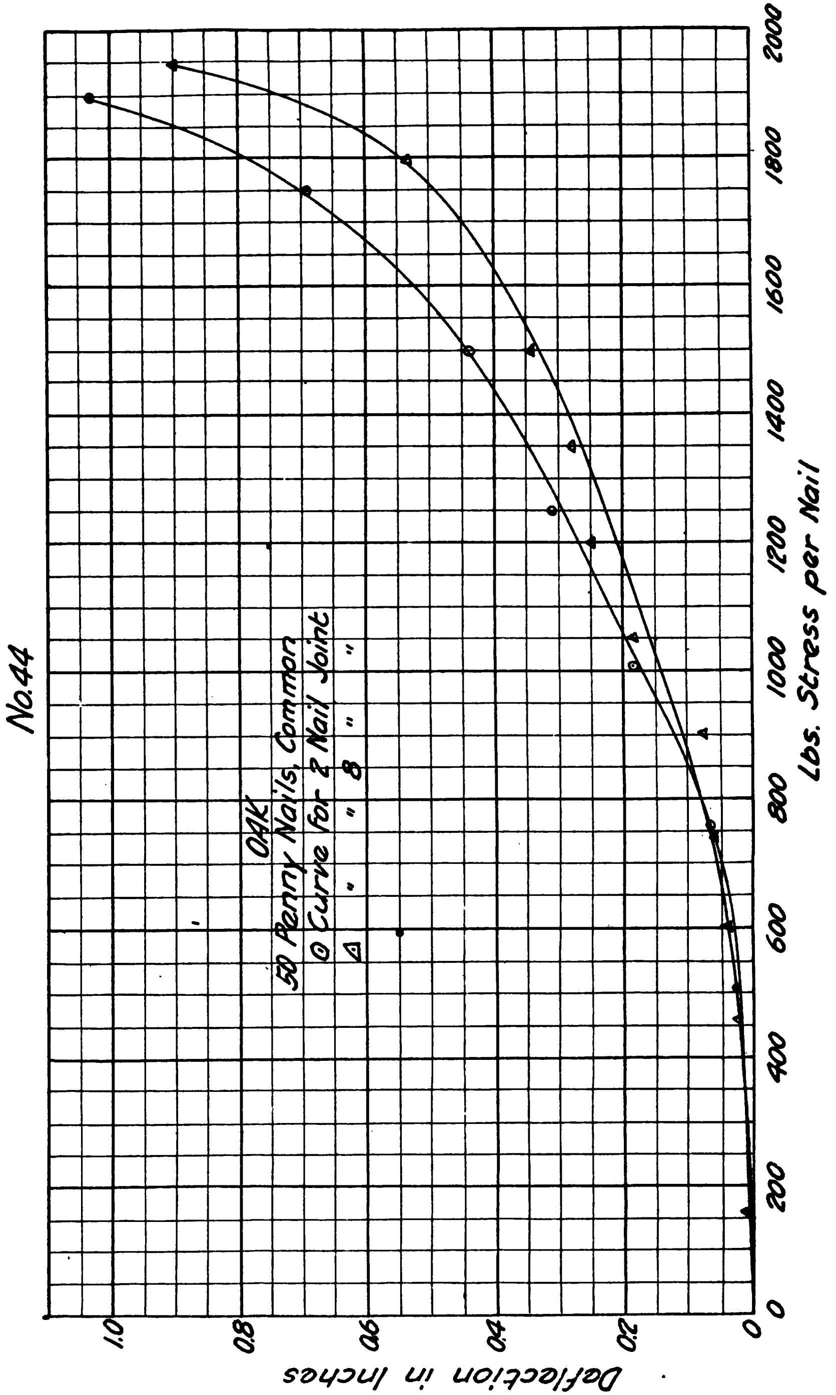

WHITE PINE
50 Penny Nails, Common
x Curve for 4 Nail Joint
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0
200
400
600
800
Lbs. Stress per Nail


No. 43
SPRUCE
50 Penny Nails, Common
⊙ Curve for 2 Nail Joint
× " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0
200
400
600
Lbs. Stress per Nail

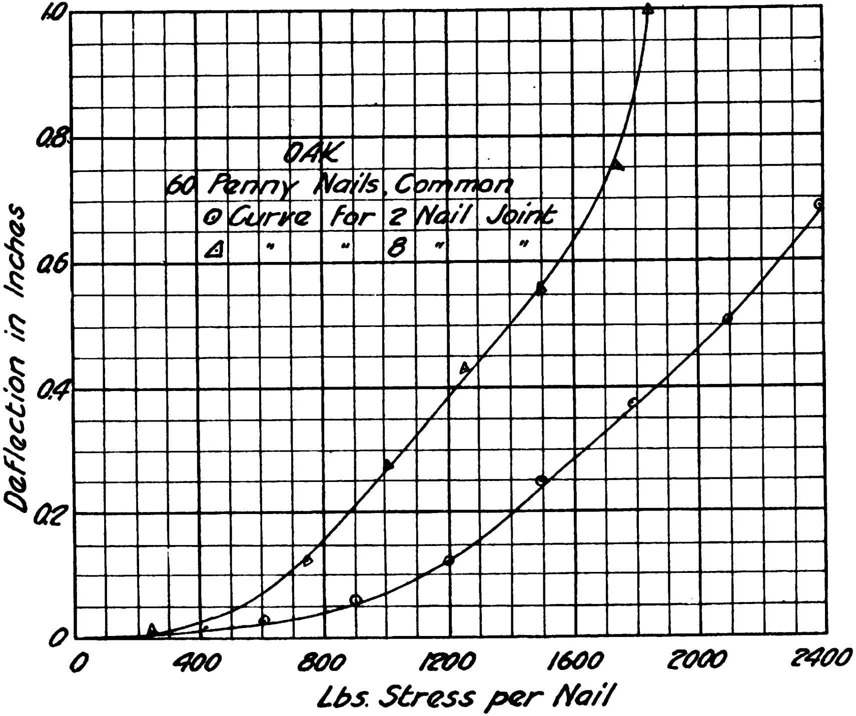
No. 42.


WHITE PINE
50 Penny Nails, Common
x Curve for 4 Nail Joint
□ " " 6 " "
△ " " 8 " "
Deflection in Inches
0
0.1
0.2
0.3
0.4
0.5
0
200
400
600
800
Lbs. Stress per Nail

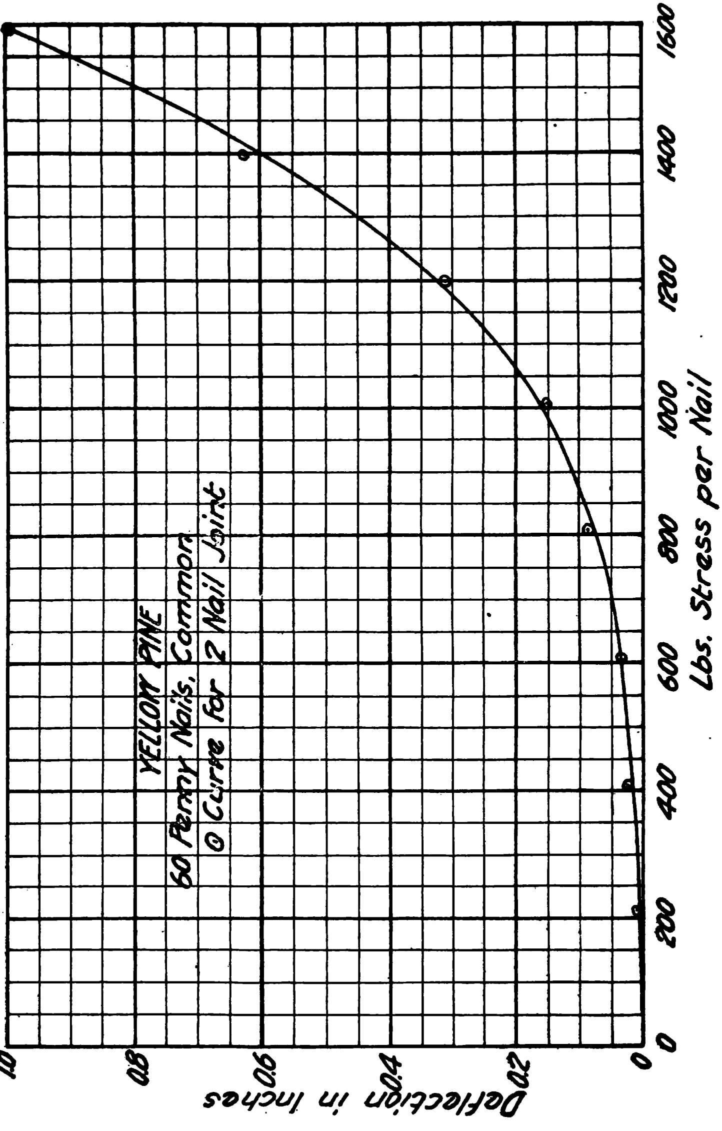

No. 43
SPRUCE
50 Penny Nails, Common
⊖ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Deflaction in Inches
0
200
400
600
Lbs. Stress per Nail


No.44
OAK
50 Penny Nails, Common
⊙ Curve for 2 Nail Joint
△ " " 8 " "
0
200
400
600
800
1000
1200
1400
1600
1800
2000
Lbs. Stress per Nail
0
0.2
0.4
0.6
0.8
1.0
Deflection in Inches

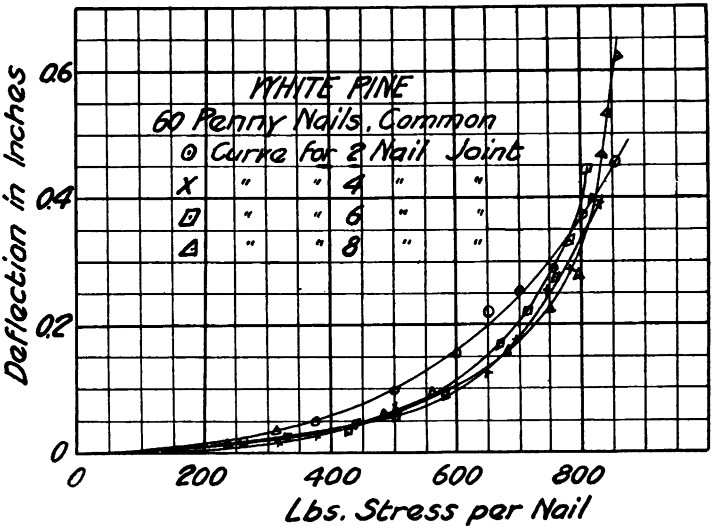
No.45


OAK
60 Penny Nails, Common
⊙ Curve for 2 Nail Joint
△ " " 8 " "
Deflection in Inches
1.0
0.8
0.6
0.4
0.2
0
0
400
800
1200
1600
2000
2400
Lbs. Stress per Nail

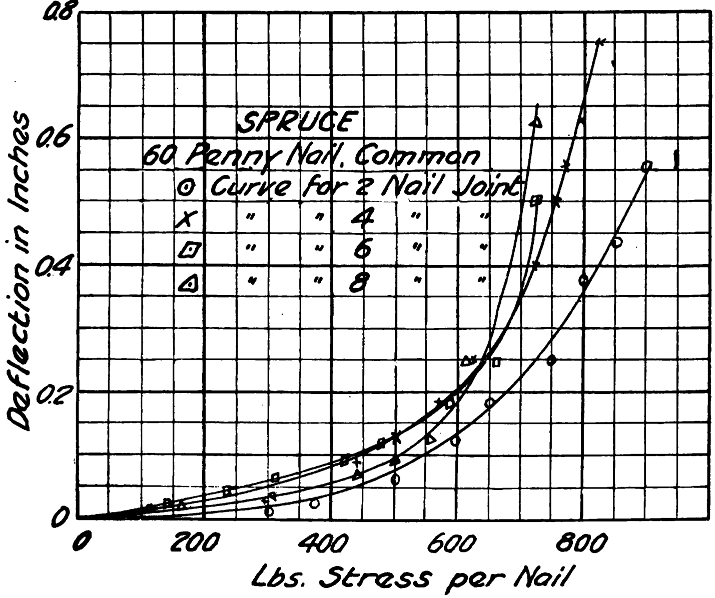
No. 46


YELLOW PINE
60 Penny Nails, Common
⊙ Curve for 2 Nail Joint
Lbs. Stress per Nail
0
200
400
600
800
1000
1200
1400
1600
Deflection in Inches
0
0.2
0.4
0.6
0.8
1.0

No.47


WHITE PINE
60 Penny Nails, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.6
0.4
0.2
0
0
200
400
600
800
Lbs. Stress per Nail

No.48


SPRUCE
60 Penny Nail, Common
⊙ Curve for 2 Nail Joint
X " " 4 " "
⊡ " " 6 " "
△ " " 8 " "
Deflection in Inches
0.8
0.6
0.4
0.2
0
0
200
400
600
800
Lbs. Stress per Nail

Engineering Experiment Station

IOWA STATE COLLEGE

Ames, Iowa

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Professor L. B. Spinney .... Electrical Engineering
Professor S. W. Beyer .... Mining Engineering

BULLETIN

Vol. IV No. III

Effect of Coloring and Water Proofing on the Strength and Permeability of Cement Blocks.

By
B. R. Smith H. L. Christian

The Improvement of Cement Mortar by Grading the Size of the Sand.

by
F. E. Cave G. H. Mack

Tests of the Impermeability of Reinforced Concrete Pipe.

By
W. A. Burton

AUGUST 1908

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O. U. MIRACLE AWARD

The three papers which are printed in this paper were prepared as theses for the degree of B. C. E. at the Iowa State College, June, 1908, and were submitted for the O. U. Miracle Award competition, for prizes offered by Mr. O. U. Miracle of Minneapolis, Minnesota, for the most meritorious papers on cement and concrete subjects prepared by students of the Iowa State College.

The President of the Iowa Cemnet User's Convention (Geo. H. Carlon, Oskaloosa, Iowa,) Mr. R. G. Coutts, Grinnell, Iowa and the Dean of the Division of Engineering of the Iowa State College were the judges, who awarded the prizes as follows:

First Prize $50.00—Effect of Coloring and Waterproofing on the Strength and Permeability of Cement Blocks.

B. R. Smith H. L. Christian

Second Prize $30.00—Tests of the Impermeability of Reinforced Concrete Pipe.

W. A. Burton.

Third Prize $20.00--The Improvement of Cement Mortar by Grading the Size of Sand.

F. E. Cave G. H. Mack

All of the above papers are the results of original investigations carried on under the direction and largely at the expense of the Engineering Experiment Station of the Iowa State College. It was thought that they merited publication herein.

However, the Engineering Experiment Station disclaims responsibility for the opinions expressed by the authors.

It is felt that while the number of tests is too small to warrant any sweeping general conclusions, yet that data on the subjects investigated are very badly needed, and that the results obtained should be put in available form.

The work has much of suggestive value, including in the case of the papers on waterproofing, especially the use of commercial waterproofing compounds, of soap and alum, and of very finely ground cement, as means of securing impermeability of cement mortars.

In the case of the paper on grading the sizes of sand for cement mortars the value of having possibilities in this line tested at the Engineering Experiment Station is shown. Any cement user or owner of a gravel pit, can send a sample, (100 pounds) to the Engineering Experiment Station, Ames, Iowa, where at a very moderate cost it will be thoroughly tested, and the possibilities of improvement by grading determined and reported.

EFFECT OF COLORING AND WATERPROOFING ON THE STRENGTH AND PERMEABILITY OF CEMENT BLOCKS.

By

B. R. SMITH H. L. CHRISTIAN

Stone, brick, and wood have been used since time immemorial in the construction of buildings. Stone has been much used for large structures because of its durability, but, except in favorable localities, its cost prevents its general use in ordinary buildings. A plain stone building is unpleasing to the eye and any ornamental cutting adds greatly to the cost. Brick is cheaper than stone but the prohibitive cost of skilled labor makes it anything but a cheap building material. Ornamental brickwork is very limited in its scope anl its cost prevents its general use.

The cost of brick and stone has generally limited the man of ordinary means to the use of wood as a building material. Wood is even more limited than brick as a sightly building material. It is far from being durable, must be kept in constant repair, and a frame dwelling is in constant danger from fire. In addition to these drawbacks the price of lumber has gone up, due to the wanton destruction of our forests, until now a small frame cottage costs as much as did a mansion some years ago.

The high price of building material, coupled with the many disastrous fires of recent years has led to the search for a cheap, fireproof material. Glass, paper, sawdust, steel, cement have all been tried; but cement is the only one that has proved practicable. It is cheap, fireproof, and besides can be made highly ornamental at but little or no additional cost. The cement stone, even the most ornamental can be made by unskilled labor, and furthermore, can be made on the spot where it is to be used. Because of these distinct advantages, cement blocks are growing rapidly in popularity until they now bid fair to rank among the most important building materials.

Two things have hindered their progress—their color and the ease with which moisture penetrates them. With these two faults overcome, cement blocks will make an ideal building material—cheap, fireproof, ornamental, beautiful. waterproof. With these qualifications they will have most of the good points of stone, brick and wood, and few of their faults.

To investigate the possibility of successfully waterproofing and coloring cement blocks was the object of this thesis. Although only a limited test was possible, enough data has been collected to convince us that cement blocks can be both colored and made waterproof.

TESTS.

For each of the different mixtures used, three separate tests were made—tensile strength test on briquettes; a transverse strength test on blocks, and a spray absorption test on blocks. In making these tests, or rather the last of the above, we were handicapped by the fact that no standard had been adopted. Mr. M. J. Reinhart, formerly of the Iowa State College, had made some tests along this line and his general ideas were followed. It was aimed to make these tests in such a manner that it could be made a standard.

While the results of some of these tests may run high or low as compared with other sets of tests, they were all made under the same conditions and so give good comparisons.

TESTING APPARATUS

The briquettes were tested for tensile strength in a standard Fairbanks testing machine.

The blocks were tested for transverse strength on a 100, 000 pound Transverse Testing Machine. Grooved contact plates, resting on round bars, carefully oiled, were used. A span of twelve inches between the supports was used in these tests.

For the spray absorption tests it was necessary to design a machine and Mr. Reinhart's machine used in the tests previously mentioned was used as a basis for experiments. To prevent the spray from striking anything but the face of the blocks, a frame was constructed. The bottom of the frame was a 2x10 plank laid flat. The top was a 2x10 plank on edge with a 2x4 spiked to the lower front face, for the bolts. The ends were of 2x10 plank set in grooves in the top and bottom plank and of such a length as to fit the block. One end was fastened rigidly to the the bottom plank and the other was left loose so it could be wedged tight after the blocks were in place. This frame was made long enough to hold three blocks. Top and bottom were clamped together with half inch bolts which could be tightened after the blocks were in place. As a packing around the blocks to prevent leakage, cotton hose, coated with white lead was used. Where blocks were chipped on the front edge the holes were filled with putty. In front of each block was swung, by a system of pipes, a section of 2-inch pipe two feet long. The front of this pipe had two rows of holes about half an inch apart. The holes were one-sixteenth of an inch in diameter and staggered one inch apart. At the back of the pipe was a stop cock for attaching a guage for testing the pressure. On the inlet pipe was another stop cock for regulating the pressure. The pressure was kept at a two inch head and the blocks were placed close enough to have the entire face kept wet.

The blocks were put in the frame, the joints made waterproof and the spray started. It was allowed to run two hours. Time was taken when the first water showed through the different partitions of the block. In no case did it go entirely through the block in two hours. The blocks were weighed before and after testing and the added weight, due to absorbed water, was figured as a percent of the weight of the block and placed in the tables.

CEMENT.

The cement used was Eastern Atlas Portland Cement and was tested according to the specifications of the American Society of Civil Engineers. All tests showed that the cement was well up to the specifications.

CHEMICAL ANALYSIS OF CEMENT

This chemical analysis and the one on clay were made by Mr. C. E. Ellis, chemist for the Iowa State College Engineering Experiment Station.

SiO_2	21 08
Al_2O_3	8.53
Fe_2O_3	3.97
CaO	61.40
MgO	1.11
K_2O	.95
Na_2O	.28
SO_3	1 52
CO_2, N_2O, etc	1.50
	100.34

SAND.

The sand used was clean, sharp and coarse. It had 35 per cent of voids and contained 3 per cent of dirt and clay. In making briquettes, only that passing a No. 8 sieve was used. In making blocks, only that passing a No. 4 sieve was used.

COLORING MATTER

The coloring matter used was a mineral color bought from stock of the Miracle Pressed Stone Co., of Minneapolis. The coloring matter was finer than the ground clay used.

ALUM.

The alum used was ordinary commercial, pulverized alum.

CLAY.

The clay used was a good clean clay, free from sticks, stones, sand, and earth. It was ground in a ball mill until at least 98 per cent would pass through a No. 200 sieve.

Its chemical analysis as determined by C. E. Ellis, is as follows:

SiO_2	63.00
Fe_2O_3	4.93
Al_2O_3	14.52
CaO	7.20
MgO	.43
K_2O	1.84
Na_2O	1.19
CO_2, etc	7.11
	100.22

BRIQUETTES.

The briquettes were made in standard molds and according to the specifications of the American Society of Civil Engineers. They were made in a room kept at about 70 degrees Fahrenheit. They were kept under damp cloths for twenty-four hours, then placed in water and kept there until tested. Sixty neat cement briquettes were made and one hundred and twenty of the others. Ten of the neat briquettes were broken at each age and twenty each of the other sets.

CONCRETE BLOCKS.

The blocks were made in a Miracle Double Staggered Air Space Machine, 1903 Model. In making these blocks no attempt was made to be scientifically accurate, although ordinary precautions were taken, but rather to follow out commercial conditions. When the clay or coloring matter was used it was first mixed well with cement. The cement and sand were mixed dry and the water added. (The sand was not dried out. Dry meaning without the addition of water.) The mixture was made just wet enough to hold its shape when squeezed in the hand then gently rolled on the palm of the hand. Each block was mixed separate and by hand. The mixture was tamped thoroughly in the mold. The blocks were made with a rough ashlar face as it was thought this would give the most severe test.

The blocks were made in January but were made in a room kept at about 70 degrees. They were thoroughly sprinkled twice a day for a week and then once a day for two weeks more. They were tested when thirty days old. They were dry when put into the spraying machine and were allowed to dry out again before being subjected to the transverse strength test. They were about thirty-seven days old when the transverse strength test was made. Five blocks of each mixture were made—three being tested at this time and two being left for tests at some future date.

The asterisk (*) indicates that the water did not penetrate this wall in the two hours given in the test. In columns where the asterisks appear it was impossible to give averages.

The blocks made with Miracle's R. O. U. Waterproofing did not show through the first face in two hours and on investigation it was found that the water, beating on the face, did not penetrate one-eighth of an inch.

Only three briquette tests have been made at this writing, seven day, twenty eight day and ninety day. Three more tests will be made—six months, one year and two years.

It was feared when these tests were started that if a waterproofing compound could be discovered, it would water-

TENSILE STRENGTH OF BRIQUETTES

Description		AGE		
		7 Days	28 Days	90 Days
14. Neat Cement made with 24% water	Max.	585	1025	830
14.	Min.	460	635	560
14.	Avg.	518	767	700
15. 3 sand 1 cement with 10% of water.	Max.	225	295	340
15.	Min.	75	150	165
15.	Avg.	153	206	281
16. 3 sand 1 (95% cement 5% clay) with 10% of water.	Max.	230	360	425
16.	Min.	120	135	125
16.	Avg.	165	231	292
17. 3 sand 1 (90% cement 10% clay) with 10% of water.	Max.	230	330	365
17.	Min.	110	160	150
17.	Avg.	169	234	279
18. 3 sand 1 (85% cement 15% clay) with 10% of water.	Max.	240	265	355
18.	Min.	105	145	155
18.	Avg.	146	208	246
20. 3 sand 1 (90% cement 10% clay) 1% brown coloring material, with 10% of water.	Max.	215	270	350
20.	Min.	120	165	165
20.	Avg.	164	211	260
21. 3 sand 1 (90% cement 10% clay) with 10% of water containing 2% of alum.	Max.	185	275	330
21.	Min.	120	170	230
21.	Avg.	154	213	281
22. 3 sand 1 cement with 10% of water containing 1% of alum.	Max.	205	300	350
22.	Min.	100	200	205
22.	Avg.	174	240	279
23. 3 sand 1 cement with 10% of water containing 2% of alum.	Max.	195	305	380
23.	Min.	140	170	220
23.	Avg.	165	233	299
24. 3 sand 1 cement with 10% of water containing 3% of alum.	Max.	210	280	325
24.	Min.	150	170	220
24.	Avg.	175	240	284
25. 3 sand 1 cement with 10% of water containing 2% of alum, and 1% of brown coloring material.	Max.	225	310	380
25.	Min.	135	170	240
25.	Avg.	189	237	321

PERMEABILITY OF BUILDING BKOCKS.

	Description	Time to Penetrate		Absorption in 2 hrs.	Transverse Modulus of Rupture
		1st Partition	2nd Partition		
		Min.	Min.	Per Cent	Lbs. Per Sq. In.
1.	3 sand, 1 cement, with 10%	11	40	4.97	242
2.	of water.	5	35	2.34	214
3.		11	65	3 84	188
	Average	9	47	3.72	215
4.	3 sand 1 (95% cement 5%	5	45	2.42	326
5.	clay) with 10% of water.	5	105	2.19	260
6.		6	35	2.11	203
	Average	5.3	62	2.24	263
7.	3 sand 1 (90% cement 10%	25	50	2.15	214
8.	clay) with 10% of water.	9	55	1.99	292
9.		19	33	2.13	188
	Average	17.6	46	2.09	231
10.	3 sand 1 (85% cement 15%	7	35	1 68	226
11.	clay) with 10% water.	45	*	.83	319
12.		7	105	1.58	225
	Average	19.6		1.36	257
13.	3 sand 1 (90% cement 10%	10	60	2.67	254
14.	clay) & 1% brown color-	25	*	1.18	306
15.	ing material with 10% of	35	*	.61	335
	water.				
	Average	23.3		1.49	298
16.	3 sand 1 (90% cement 10%	67	*	.72	355
17.	clay) with 10% of water	6	100	1.67	231
18.	containing 2% of alum.	20	110	1.03	317
	Average	31		1.14	301
19.	3 sand, 1 cement with 10%	6	36	1.92	364
20.	water containing 2% alum	7	70	1.65	393
21.		17	90	.41	418
	Average	10	65	1.33	392
22.	3 sand, 1 cement with 10%	17	*	.61	268
23.	of water containing 2%	30	*	.31	280
24.	alum.	10	*	.60	315
	Average	19		.51	288
25.	3 sand, 1 cement with 10%	6	*	.55	328
26.	water, 3% alum.	5	*	.10	294
27.		9	*	.10	358
	Average	6.6			327
28.	3 sand, 1 cement, 10% water	15	*	.60	344
29.	containing 2% alum 1%	48	*	.20	269
30.	coloring matter.	21	*	.61	278
	Average	28		.47	297
31.	3 sand, 1 cement washed	10	*	.65	242
32.	with Winslows Hydro-	*	*	.11	214
33.	lithic Coating.	9	*	.11	188
	Average			.29	215
34.	3 sand 1 cement from ½-2%	*	*	.10	412
35.	R. O. U. Water-Proof-	*	*	.10	360
36.	ing.	*	*	.10	440
	Average			.10	404

*Did not penetrate partition.

proof the block so quickly that the daily wettings while curing would be unable to penetrate it and thus the middle would not be given sufficient water to properly set. From the tables of results it will be seen that the blocks nearest waterproof are the strongest.

Although good results were obtained on these tests and although we are convinced that the results would give a good indication of what the final results would be, we reailze that the tests covered too limited a period of time to be taken as absolute. To completely cover the subject would require tests covering years. There are many points that were not taken up in this subject but so far as possible the essentials were covered.

The coloring matter used held its color well even after being immersed in water for ninety days. The fracture of the briquettes showed an even color.

The alum bleached the blocks slightly. The blocks without coloring matter showed light gray in place of blue gray.

Two patent waterproofings were used, E. J. Winslow's Hydrolithic Coating and Miracle's R. O. U. Waterproofing. The Hydrolthic coating was applied to the face of the three sand-cement blocks previously tested and found to be far from waterproof. The R. O. U. was mixed with the cement, it being a dry powder, when the blocks were made.

Clay or clay and coloring matter had little or no effect on the strength of the briquettees. The five and ten percent of clay, although replacing cement, seemed to add a little to the strength, showing 231 pounds and 234 pounds per square inch respectively. The fifteen percent of clay and the ten percent of clay with one percent of coloring matter (one per-cent of the total equals about four per cent of the cement) give about the same results as sand and cement, 208 pounds, 211 pounds and 206 pounds respectively. At ninety days the clay seemed to have reached its limit and as compared with the sand cement briquette is not as strong as at twenty-eight days. The plain cement-sand, the ones with five percent, ten percent, fifteen percent of clay and ten percent of clay and one percent of color running as follows: 281 pounds, 292 pounds, 279 pounds, 246 pounds, 260 pounds.

The stress per square inch in the blocks runs higher than in the briquettes. The blocks containing clay, and clay and color did not soak up as large a percent of water as did the sand cement block. Sand-cement block 3.72 percent; five percent of clay 2.24 percent; ten percent of clay 2.09 percent; fifteen percent of clay 1.36 percent; ten percent of clay and one percent of color 1.49 percent. Apparently the more clay the more nearly waterproof. Again coloring matter seemed to act on a par with clay.

In the briquettes ten percent of clay and water containing two per cent of alum showed about the same strength as sand-cement alone. In the block it showed stronger, 301 pounds against 215 pounds. It soaked up 1.14 percent against 3.72 percent. From these results it would appear that the clay helped the strength of the cement for a short time but after that was a detriment—how much, later tests will show. Its addition made the blocks more waterproof than before but they still took up too much water.

Alum added to the water used in mixing the briquettes seemed to have but little effect on the briquettes, the results of the alum water briquettes and the sand-cement briquette tests running quite parallel. The strength as shown in the blocks is increased; the sand-cement showing 215 pounds per square inch as against 392 pounds, 288 pounds, 327 pounds and 297 pounds. In waterproofing is where the alum shows its worth. In the case where the water used contained three percent of alum the block was nearly waterproof. The block mixed with one percent alum solution soaked up 1.33 percent; the two percent alum solution block soaked up .51 percent. The three percent solution block soaked up .25 per cent. The two percent alum solution in the block with one percent of coloring matter soaked up .47 percent.

From this it would seem that while alum does not affect the strength to any great extent, it makes the cement almost impervious to water.

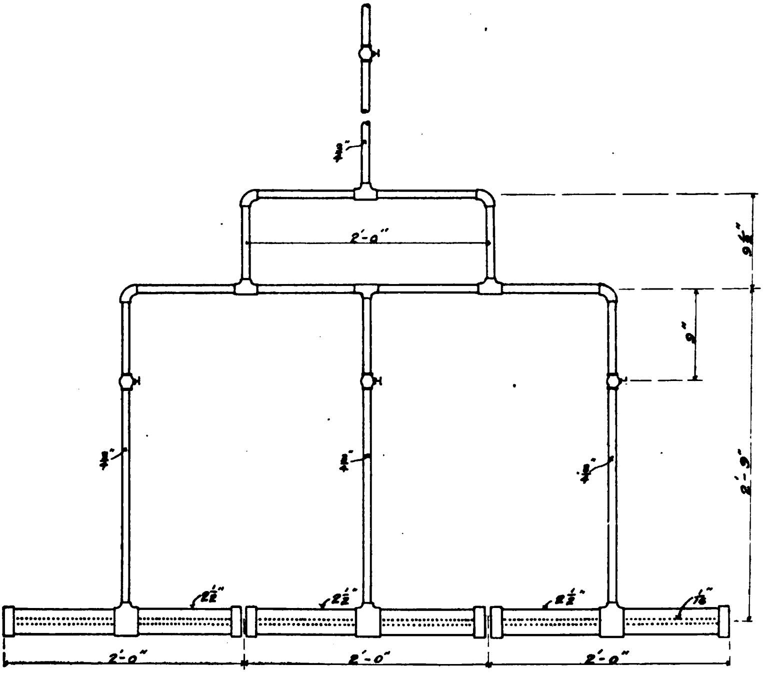
Winslow's Hydrolithic Coating had about the same effect on the blocks as the three percent alum solution. The blocks that soaked up 3.72 percent only took up .29 percent when coated with this mixture.

Miracle's R. O. U. Waterproofing gave the best results obtained. In two hours the water did no show through even

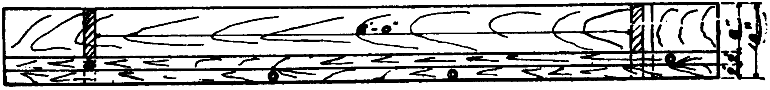
(See Next Page)

the first partition although the spray was playing directly against the opposite face. On breaking the blocks immediately afterward it was found that the water had not penetrated one-eighth of an inch. The blocks absorbed only .01 percent and the modulus of rupture was 404 pounds per square inch.

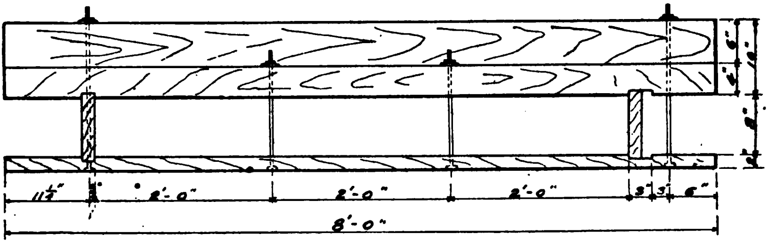
From these facts it will be seen that the blocks can be colored and waterproofed without hurting the early strength at least. From an inspection of the results it would seem that longer tests would show no different results. However, time will tell.



DESIGN
OF
BLOCK SPRAYER

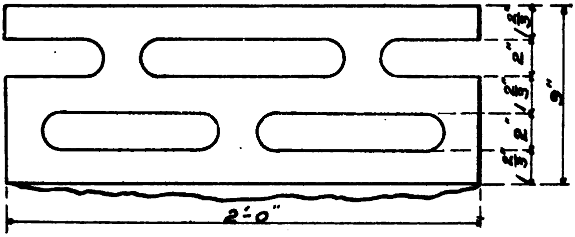


PLAN



ELEVATION

DESIGN
OF
.BLOCK RACK.



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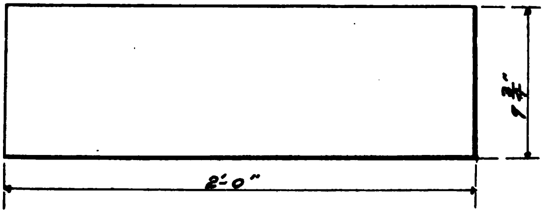


ELEVATION

DESIGN
OF
.CONCRETE BLOCK.

TESTS OF IMPERMEABILIAY OF REINFORCED CONCRETE PIPES

By

W. A. BURTON

The rapid development in the use of concrete for so many different kinds of structures has brought out many of its merits, but at the same time, some of its unsatisfactory qualities have come up for investigation. Numberless tests have been made on reinforced concrete beams for cross bending, and on the tensile and compressive stresses of concrete.. There are comparatively few records of tests of the impermeability of concrete. Some of the most important that were found in engineering literature are here given.

The following is taken from Buell and Hill's Reinforced Concrete: "Mr. Feret's experiments showed that for maximum impermeability it is best to use too much rather than too little water. Specimens showed that the permeability of mortars subjected to the infiltration of fresh or sea water rapidly decreases with time."

J. B. McIntire and A. L. True of the Thayer School of Engineering conclude as follows from experiments for a thesis "All specimens composed of 1:1 mortars in proportions of 30 percent to 45 percent of the whole mass were impermeable at pressures somewhere between 20 and 80 pounds per square inch."

W. K. Hatt with refernce to soap and alum says: "Soap and alum in sand mortar decreases absorption 50 percent without greatly affecting the strength."

Experiments made at a later date than any of the above were made by Wm. B. Fuller and Sanford E. Thompson. The results of these experiments were given in a paper read before the American Society of Civil Engineers in March 1907. The authors make these observations and conclusions:

"The results of the permeability tests emphasize the fact that little is known of the action of concrete in resisting the flow of water.

Permeability decreases with age, increases nearly uniformly with the increase of pressure and increases as the thickness decreases, but in a much larger inverse ratio. The tests were made upon blocks 6x6x17 inches taken from concrete beams previously tested in cross bending and the test was so made that the water had to pass the length of the block or 17 inches.

The Reclamation Service has tested Reinforced Concrete Pipes of structural size under heads of water up to 50 pounds per sq. in. with results fairly satisfactory.

Most of the tests thus far enumerated have been on specimens not containing any waterproofing, reliance being placed on the richness of the mixture to produce impermeability. The one exception is that in which soap and alum were used.

Mr. R. H. Gaines recently made some experiments for the New York Board of Water Supply in which clay was used as a water proofing. No standard of testing has been adopted, each experimenter following his best judgement under the conditions.

It will be seen that pressure and thicknes of concrete are two important factors to be considered in permeability tests. The thickness of the specimens tested for this thesis was 4 inches, not allowing for inaccuracy in forms, which in every case would tend to reduce it. The inside diameter and length of specimen given, the water passing per unit area can be computed but except in cases of much leakage, which are not important anyhow, this refinement would not be of much value.

OBJECT OF TESTS.

The object of these experiments was to discover a concrete four inches thick (about the minimum for structural purposes) which would be impermeable to a head of water of 100 feet, the same not to be impracticable on account of expense. Incidentally it was desired to find out the effect of water pressure on joints in concrete work, i. e. where new concrete is built against old.

IMPORTANCE OF IMPERMEABILITY

As to the importance of this subject it may be said in a general way that it is desirable that nearly all concrete should not allow water to pass through it. Concrete blocks are not entirely satisfactory because of the dampness of buildings which they are made. All reinforced concrete structures exposed to rain or water are likely to be injured if much water comes in contact with the reinforcing. While the CO^2 of the cement does much to prevent the steel from rusting, it will not do to rely upon it against an excess of water. Hence all reinforced arches, slab bridges and roofs of concrete are at least of doubtful durability if a high degree of impermeability is not attained.

To come to special cases where water tightness is absolutely necessary, there may be mentioned conduits, reservoirs and tanks and pressure pipes. All of these fail entirely of their purpose if leakage is not reduced to a negligible quantity.

Sewers made of concrete should be practically impermeable in order to prevent sewage escaping and contaminating wells or otherwise causing a nuisance. Many of these results have not been attained. The Engineering News in an editorial says: "We often allow water to percolate through concrete simply because we cannot help ourselves."

MATERIALS USED

CEMENT. The cement used was the Atlas brand and was found to pass all the requirements specified by the American Society for Testing Materials.

SAND. The sand for the specimens was obtained from the same source as that used around the college for floors, sidewalks and other concrete work. As to coarseness, that which passed a No. 8 sieve was considered sand and that retained on a No. 8 sieve was considered gravel. The following is the result of a fineness test, 100 grams being taken.

Grams	retained on	No. 10	sieve	11.7
‘‘	‘‘	No. 20	‘‘	23.0
‘‘	‘‘	No. 30	‘‘	19.
‘‘	‘‘	No. 40	‘‘	16
‘‘	‘‘	No. 50	‘‘	11.
‘‘	passing on	No. 50	‘‘	18.
	Total			99.4

It is thus seen that the sand was pretty well graded. The voids in the sand as determined by several different tests averaged close to 35 percent.

The amount of *suspended matter,* mostly clay, was determined by carefully drying a quantity of sand, weighing it, then washing it in a shallow vessel, agitation the sand while the water flows slowly through it and out over the top. When the water flows clear, the sand is taken out, dried and weighed. The difference in weight divided by the weight of the sand before is the ratio of suspended matter. It was found in this sand to be 2 percent.

GRAVEL. The gravel, as before stated was obtained by screening the sand and using what was retained on a No. 8 sieve. The bulk of the pebbles were between 1-8 inch and 1 inch in diameter, not many being more than 1-½ inches in diameter.

STONE. The crushed stone was ordinary Iowa limestone passing a 2-½ inch screen. The crusher run was used so there was considerable stone dust present.

REINFORCING. The reinforcing used was ordinary No. 6 galvanized wire. This was cut to the proper length, hooks were made upon each end and the wire bent into circular form and the hooks engaged, thus making a good, solid connection. It was not intended to stress the concrete beyond its ultimate strength but reinforcing was put in for added safty.

SUBSTANCES USED FOR WATERPROOFING

CLAY. The use of clay for waterproofing has been suggested at times by prominent men, but owing to experiments made by Mr. R. H. Gaines and published in the Engineering News Sept. 26, 1907, it was thought well to test clay as a waterproofing material. Mr. Gaines' theory is that clay when mixed with water enters into a hydrated non-chrystalline condition known as colloid. The colloids cause the slightly soluble salts in the cement to precipitate slowly and also prevent the calcium compounds from quick hydration. This prevents any sudden changes of temperature and thus allows the cement to set with conditions favorable to a maximum density, and maximum density means maximum impermeability.

The experiments made by Mr. Gaines indicate that with 5 per cent or 10 per cent of clay in a specimen tested at 30 days under 80 pounds pressure the leakage was nil.

The clay used was such as is found in the vicinity of Ames. Clay from this pit was formerly used in making brick. The clay was dried and ground in the ball mill so fine that 98 percent would pass a No. 200 sieve. The time required to grind to this fineness was about three hours.

ALUM. The alum used was the ordinary commercial article purchased at the pharmacists.

SOFT SOAP. The soft soap was a home made article and composed of water, tallow, and potash in proportions ordinarily recommended in the directions on the potash can.

Soap and alum have long been recommended as a waterproofing so it was thought best to compare this with clay.

REGROUND CEMENT. The idea of using reground cement resulted from the fact that tension tests made by the experiment station on briquettes of reground cement showed a great increase in strength.

METHOD OF MANUFACTURE.

In making the pipes the aim was to duplicate good, practical conditions. All the concrete was mixed in a shed warmed by an ordinary stove.

MEASUREMENTS. All the materials were measured by volume except the waterproofing which was weighed and taken as percents of the cement, water, or cement and sand combined as the case might be. A box 8x8x8 inches inside was used to measure the cement, sand, gravel and broken stone.

MIXING. The cement and sand were placed in a box and thoroughly mixed dry with a shovel. Water was then added and this was mixed into a thin mortar with a hoe; finally the broken stone and more water, if necessary, was added and the whole turned over with a shovel twice. The mixing was then continued until the mass was thoroughly mixed.

WATER. Because of the observations of other experimenters on impermeability the concrete was always made moderately wet. It would always quake with a little agitation.

QUANTITY IN BATCH. Usually enough for two pipes was mixed in a batch or about 2 cu. ft., but a few times enough was mixed for four pipes.

PLACING IN MOLDS. With the exception of the first nine which were made in a shed, the pipes were made in the masonry laboratory. Hence after mixing, the concrete was wheeled from the shed to the masonry laboratory. When placing the concrete in the molds it was spaded next to the forms with a small flat strip of iron. The reinforcing was put in as shown in figure 1., six circular wires in each pipe. The top of each pipe was finished with a trowel smooth enough to make a good fitting with the cap and gasket used in testing.

FORMS. The forms on the outside consisted of a sheet of No. 16 galvanized iron bent in the form of a cylinder, and the joint secured by ½ inch x ¼ inch bolts. The inside form was the 6 inch tile mold of the Miracle Company. The molds were very satisfactory for the purpose. They were cleaned of any adhering concrete and oiled each time they were used. These molds were each 18 inches long and the outside one was 14 inches in diameter so as to make the pipe 18 inches long, 6 inches inside diameter and 4 inches thick.

STORAGE. The first nine of the pipes were made in the shed and left there for three days and four of these, two of No. 101 and two of No. 102 were slightly frozen before setting. These and all that were made afterwards were stored in the masonry laboratory until they were tested. The temperature of this room was nearly constant and 70 degress Fahr. The pipes were wet sprinkled every two days for two weeks after bing made.

APPARATUS USED FOR TESTING

This is shown in figure No. 2.

It consisted of 2-½ inch x 16 inch x 16 inch plates each containing eight holes to receive bolts for clamipng. ½ inch bolts 20 inches long were used. By figure 2 it is seen that water is introduced to the pipe by a hose and that another hose connects from the pipe to the pressure guage. A waste valve is placed between the tap and the pipe so that the pressure can be regulated as desired. The waste valve was first placed at (b) as shown in figure 2 but was removed to (a) as shown, in time to make the last few tests of table II.

In clamping the caps to the end of the pipes a ¼ inch rubber gasket is placed betwen the plate and the pipe. In some cases the pipe had to be rubbed smooth or a little cement plaster applied to the rough ends.

METHOD OF TESTING. It was difficult at the start to tell just how to begin the tests so they could be carried through uniformly. The size of each pipe is 18 inches long by 6 inches inside and 14 inches outside diameter. Thus the tension in the concrete for 60 pounds per square inch pressure = radius x pressure per sq. in ÷ thickness of pipe = 3 x 60 ÷ 4 = 45 pounds. It is not thought that there would be much trouble from breaking, but nevertheless one or two specimens broke under the tests.

Further it was not known what water pressure would best suit all of the pipes. Thus most of the pipes stood 60 pounds but some at that pressure would leak too much to measure. It was necessary to make tests that would show maximum leakage and also in time so as to test them all at the same age. In the first sets of tests the measurement of water passing in twenty minutes at the end of three hours was the standard chosen. At the end of this time the pipe would in most cases have reached its maximum leakage. Some of the best specimens may not have quite attained maximum at this time, but near enough for the purpose. In some of the special tests made later with the best pipes it appears that the maximum leakage may not come for a couple of days, but still there was not much increase after twenty four hours.

As shown in figure 2 the pipe was tested in a nearly horizontal position, but in order to get the water passing per foot of length, strings were tied around the pipe one foot apart and the water coming out between these strings was caught in a conical vessel one foot in diameter. This vessel was placed so as to catch the water which dropped from the string lowest down, but not the upper one.

In the second set of tests it was first intended to find a pressure for each kind of pipe at which there would be no leakage after twenty four hours, but this was found to involve so many guesses that instead it was decided to test at 90 days under or about the same conditions as in the first test except that the measurement was made at the end of twenty four instead of three hours. In this way any errors in the first test would be checked. In the 90 day test no discrepancies appeared. All of the best pipes tested as well or better in 90 than in the 60 day test.

SPECIAL TEST. The two best pipes, one a mixture of 1:2:3½ stone with soap and alum for water proofing and the other 1:2:3½ stone with reground cement were tested for periods of 12 days and the results are interesting in showing that both could be relied upon for almost any practical purpose where water tight construction is necessary with pressure not greater than used in the test; viz., 45 pounds.

It further appears that there is a tendency for the leakage to decrease with time. This decrease of permeability may be the result of cementation similar to that in the formation of natural rocks.

GENERAL CONCLUSIONS.

GRAVEL vs. BROKEN STONE. One fact that stands out prominently in all the tests is that broken stone is much better for impermeability than gravel. The surprise in the first set of tests, Table 11, was that the gravel mixtures could not be tested at all at 60 pounds, save one, and it showed a leakage of 86 oz. as compared with three similar pipes with broken stone as aggregate which averaged only 16 oz. In the 90 day test (Table III.) gravel mixtures at 30 lbs. pressure showed more leakage than similar broken stone mixtures at 50 lbs. pressure. It is certainly not possible that the broken stone pipes had more favorable conditions as to manufacture than the ones in which gravel was used.

CLAY. As to the value of the different methods of water proofing clay was rather a disappointment. In the first three kinds of pipes tested in Table II. having respectively clay, 5 percent and 10 percent of cement = clay, the latter quantity of clay seems to have somewhat prevented the passage of water, but pipe No. 112 with no clay, but otherwise similar was better than any one of the first three kinds. Not relying on these for comparison as all of the No. 100's, and 101's and two of the No. 102's might have been slightly injured by freezing, others made later containing clay and not so injured did not test as well as No. 112 containing no clay, but otherwise similar. Table II. shows that the tensile strength of 1:2 mixtures containing clay is materially decreased, but of 1:3 mixtures it is slightly increased. It might be said that the latter mixture is useless for watertight construction, anyway.

SOAP AND ALUM. In connection with using soap and alum for waterproof mortar Baker's Masonry states: "The alum and soap combine and form compounds of alumina and the fatty acids which are insoluble in water." On the pipes in which soap and alum were used the first tests were very encouraging. The best of these was included in the 90 day test when only a few drops came through in twenty minutes after being under 45 pounds pressure for twenty four hours. This same specimen was also kept at 45 pounds pressure for 12 days and the results given in Table IV. It shows that permeability tends to decrease with time, also that the leakage was practically nothing at the end of the test, more than 1-3 of the outside of the pipe being as dry as tinder.

The tests on the briquettes corresponding to the soap and alum pipes showed that the tensile strength is not materially affected by the addition of soap and alum. As these experiments verify many others and considerable experience, it may be considered safe practice to use soap and alum to secure impermeability and it is probable that under the same conditions of thickness of concrete, pressure, and workmanship that soap and alum mixtures similar to that used in the test and used in structures would do as well as in the test. The difficulties in using soap and alum are not great and the cost is not excessive.

REGROUND CEMENT. As before stated, tests made by the experiment station indicated that the strength of briquettes containing reground cement was considerably greater than when the same cement was used without regrinding. Both of the pipes made of (See Table I) 1:2:3½ broken stone, the cement being reground, showed less leakage than any of the rest. The permeability of these specimens at 50 pounds was practically nil and from observations of the dampness on the outside they appear better than the pipes containing soap and alum.

In a 15 day test with one of these two the leakage of the pipe and connections at cap reached a maximum of a drop about every five minutes in 24 hours, but at the end of 15 days the drops ceased to appear and the damp area was decreasing. This was at a pressure of 50 pounds.

The briquettes corresponding to these latter pipes showed a tensile strength fully 25 percent in excess of the 1:2 briquettes, ordinary cement being used, at 7 days and 60 days, and they also showed better in the 28 day test.

The substances in the latter pipe as well as the method of making them could easily be duplicated. While the regrinding of the cement in the ball mill of the masonry laboratory was a slow process, it would not be at the cement mill. Let it be known to the mills that there is a demand for a cement of a greater fineness than is now manufactured and they will soon have a stock of that kind of goods on hand. It is not known that any tests have before been made in which reground cement of the fineness here used was relied upon for impermeability, hence it would be better to await further tests before asserting anything radical. The conclusions to be drawn from the tests are unmistakable and indicate that cement of the fineness of that used in these experiments would be the safest thing to try for impermeability under high pressure.

TABLE NO. I.

Specimen No.	number made	Proportions				Water Proofing			Remarks
		Cement	Sand	Gravel	Broken Stone	Fine Clay	Potash Soap	Alum	
100	3	1	2		3½				
101	4	1	2		3½	5%			
102	4	1	2		3½	10%			
103	4	1	2	3½		5%			
104	2	1	3	5		5%			
105	2	1	2		3½		1% water = soap	1% sand plus cement equals alum	
106	2	1	3	5		10%			
107	2	1	2	3½		10%			
108	3	1	2		3½	5%			Joint-half pipe made one day and finished in 48 hrs
109	1	1	2		3½	5%			Collar Joint otherwise same as above.
110	2	1	2		3½				Reground cement 94% passing No. 200 sieve.
111	1	1	2			5%			
112	1	1	2		3½				

TABLE NO. 2. TENSION TEST.

Lbs. per sq. in .... Five specimems averaged.

Proportions		Water Proofing	7 Day	28 Day	60 Day
Cement	Sand				
1	3			271	301
1	3	5% Clay	161	149	313
1	3	10% "	196	298	342
1	2	Soap and Alum	245	330	400
1	2	Reground cement	360	427	513
1	2	None	247	379	407
1	2	5% Clay	240	361	356
1	2	10% "	291	397	365

Note:—Table No. 2 continued on following page.

TABLE NO. II. (Continued). 60 DAY IMPERMEABILITY TEST.

Pipe No.	Proportions	Waterproofing	Pressure lbs. per sq. in.	Water pas-ing last 20 minutes of 3 hours per foot of pipe. Ozs. per specimen	Water pas-ing last 20 minutes of 3 hours per foot of pipe. Average	Remarks
100	1:2:3½ stone	None	55	120		
100	" "	"	60	12	66	Specimen had been under pressure before.
101	" "	5 per cent Clay	65	35		
101	" "	5 " "	65	88		
101	" "	5 " "	65	154	92	
102	" "	10 " "	60	12		One end frozen and leaked badly; measured leakage of good end.
102	" "	10 " "	60	10		
102	" "	10 " "	60	27	16	
102	" "	10 " "				Spurted out too much to measure
103	" gravel	5 " "	60	Too much to measure		
103	" "	5 " "	60			30 pounds pressure caused little leakage.
107	" "	10 " "	60	86		
104	1:3:5 "	5 " "	30	40		
104	" "	5 " "	30	20	30	
106	" "	10 " "	30	18		
106	" "	10 " "	30	15	17	
105	1:2:3½ stone	Soap and Alum as per Table I.	60	7		Scarcely leaked at all first hour.
105	" "	do.	60	.5	3.75	Filled with water since it was made. ⅛ outside surface wet at end of test.
110	" "	Reground cement-94% passing No. 200 sieve.	60	0		Not wet on outside except near end.
110	" "		60	2.5	1.25	Plane joint. Did not apparently leak more at joint than elsewhere.
108	" "	5 per cent Clay	60	8		
108	" "	5 " "	Too much to measure			Plane joint; leakage excessive at joint.
108	" "	5 " "	60	10		Plane joint.
109	" "	5 " "	60	17	12	Collar joint. Part of joint still dry.
112	" "	None	60	2	2	
111	1:2	5 per cent Clay	60	2	2	Burst at end of experiment.

TABLE NO. III. 90 DAY IMPERMEABILITY TEST

Pipe No.	Proportions	Waterproofing	Pressure lbs. per sq. in.	Water passing last 20 min. of 24 hrs. per ft. of pipe.		Remarks
				Ozs. per specimen	Average	
Best						
104	1:3:5 gravel	5% clay	30	20	20	
106	1:3:5 "	10% "	30	14	14	
107	1:2:3½ "	10% "	30	9		
107	1:2:3½ "	10% "	30	4	6.5	
108	" stone	5% "	50	5		Plane joints
108	" "	5% "	50	2	3.5	Joint not leaking more than elsewhere.
Best						
102	1:2:3½ stone	10% "	50	6	6	Tested only 4 hrs., but was all wet on outside.
110	" "	reground cement	50	0		Few damp spots on outside.
110	" "	" "	50	0	0	
105	" "	soap & alum		4 drops		Part of 12 day test.
112	" "	none	50	.5	.5	Last one made and special care taken.

TABLE NO. IV. SPECIAL TESTS.

Specimen No.	Date	Temperature Fahr.	Pressure lbs. per sq. in.	Total water passing fnll length pipe. Drops per min.	Remarks
Best 105 Soap and Alum see Table 1.	Begun test April 1, 1908		44	8	
	" 2 "		44	10	
	" 3 "		42	10	Damp area increasing.
	" 4 "	78	45	10	
	" 6 "	74	45	8	
	" 7 "	70	45	5	
	" 8 "	66	45	5	
	" 9 "	66	45	4–	
	" 10 "	66	43	2 ½	
	" 11 "	76	45	2	
	" 13 "		42	1	About ⅓ outside surface never wet.
110 Reground cement Table 1.	Began Test May 4, 1908	Close to 70 all the time	45	Drop about every 10 minutes	
	" 8 "		50		One end dry, other wet, mark 6 feet from end.
	" 11 "		45		Damp area constant.
	" 18 "		45	Not a drop in sight	Damp area much less.

Note:—In these tests pressure was kept on continuously day and night.

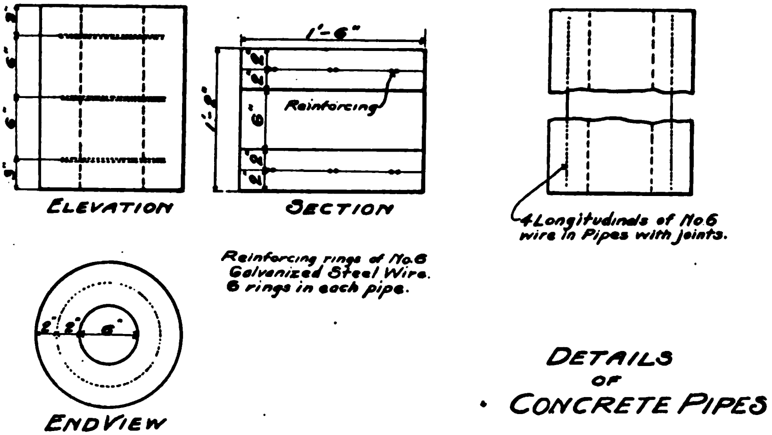


FIGURE I.

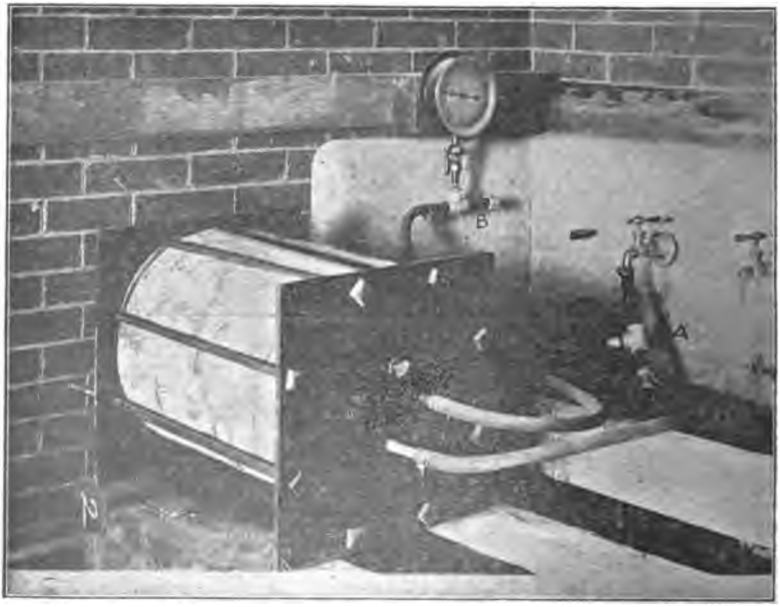


FIGURE II.

THE IMPROVEMENT OF CEMENT MORTAR BY GRADING THE SIZE OF SAND.

By

F. E. Cave G. H. Mack

Sand is used with cement to reduce the cost of the mortar and also because neat cement will crack. Pulverized stone cinders, or slag may be used, but sand is much the most common. For work of large proportions sand is used on account of its wide distribution and consequent cheapness.

More attention should be paid to the examination and testing of the sand available. It is usually the practice to use the sand most easily obtained without regard to its condition other than cleanliness.

The smaller the percentage of voids in a sand the less cement is needed and consequently the cheaper the mortar. In cement mortar enough cement should be used to coat each particle of sand and to fill the voids, otherwise the mortar will be porous and weak. In a fine sand of uniform grain, more cement is needed to coat each particle of sand. The proportion of voids is independent of the size of the sand but depends upon the gradation.

The object of this thesis was to determine whether or not it was possible to increase the strength of cement mortar by grading the size of sand and keep the cost within practical limits. The sand was graded into two sizes and these were mixed as explained in the following pages.

The first difficulty was to get samples from the different parts of the state. Several companies dealing in sand and gravel were requested to send samples, but Smith & Oakes of Clinton, Iowa, was the only firm that responded. Mr. Mack brought a sample from his home at Dows, Iowa, and the other three samples were obtained at Ames.

Sample No. 1. was obtained at New Agricultural Hall and was used in all the concrete floors in that building. It was from a pit southeast of the campus. It is a yellow sand and ranks second in the amount of fine material present. It was a good clean sand with medium sharp angular grains.

Sample No. 2 came from Munn and Son of Ames and is a

Squaw creek sand from south of Ames. It is not a clean looking sand but shows a small percentage of fine material. It was a dirty white in color.

Sample No. 3. came from Smith & Oakes of Clinton, Iowa and was coarse and had a small percentage of fine material. It was dark colored and was composed of a large percentage of large black grains. For further description we quote from their letter: "We are shipping to you today a sack of sand and gravel just as it is pumped from the bed of the river. We have no pit, our sand and gravel being pumped from the bed of the river. This process washes out all impurities and leaves it clean, much cleaner than any 'pit' gravel or sand."

Sample No. 4 came from the same pit as Sample No. 1. There are many different kinds of sand in this pit and No. 4 shows a distinct difference from No. 1., not only in fineness and strength, but in color, it being lighter in color.

Sample No. 5 was obtained by Mr. Mack from a pit near Dows, Iowa in a glacial mound of the type known to geologists as kame. It was light in color and the dirtiest sand of all the samples. The grains were very sharp and angular.

The sand was first sifted through the different sieves, and the percentages retained on each sieve are shown in table No. 1a. The figures include the amount retained on all the preceding sieves. Thus the amount retained on the No. 30 sieve includes that retained on sieves Nos. 20, 16, 10, 8, 4 and 2 respectively, as well as that passing the No. 20 and retained on the No. 30 sieve.

All material retained on a No. 4 sieve was discarded as gravel, and the mixtures were made up of two kinds of sand, that retained on a No. 16 and that passing a No .16 sieve.

In the determintion of voids, both the displacement and specific gravity methods were used. In the displacement method a 200 cc. graduate was filled half full of water, and 100 cc. of sand well shaken was added to it. The percentage of voids was then read from the large graduate from the top down. Determination by the specific gravity method was effected by dividing the weight of sand in a given measure by the weight of the given measure, filled with solid sand. The result substracted from 100 will give the percentage of voids.

The specific gravity of the sand was determined by the method described in the Report of Committee of American Society for Testing Materials on Standard Specifications for Cement, using Benzine.

Table No. 1a. is self explanatory as it simply shows the percentage of different sizes in each mixture.

Table No. 1b is also very simple excepting the second and third columns showing the percentage of voids. Here is an apparent discrepancy for these should give the same results. The method by specific gravity, however, in all cases gives a smaller result than by the displacement method. In the method by displacement, water was used. In the specific gravity method benzine was used in determining the specific gravity of the sand. Benzine was used because it did not become cloudy when used with dirty sand, and did not form bubbles on the surface as did the water. This matter was investigated quite thoroughly and the result of each method checked. Some experiments were made to get comparative results for specific gravity using benzine, and using water. The average of three results using benzine was 2.42, and the average of three using water was 2.61. The sand used was sample No. 5 which was the dirtiest of the lot. Although this gives a possible reason for the discrepancy of results it does not explain them fully, and the authors are unable to explain why these two methods should not give the same results.

Although results obtained by these two methods do not check, the results of several experiments by either method alone agree quite closely and it is evident that either method may be used with reasonable accuracy in securing comparative results. The displacement method was used in comparing the percentage of voids in the original sand with the percentage in the mixture.

The mixtures obtained in Table No. 2 were determined by experiment. To a certain quantity of the original sand of each sample a percentage of that amount of fine or coarse sand was added until the minimum percentage of voids was found. The mixture was then taken for use in the tests of the mixtures.

Table No. 3 shows tests of the cement used in the tests, and is self explanatory.

Table 4 and 5 show results of tests of original sand and of the practical mixtures. Owing to a lack of material present, three month tests of the first three samples were not made.

Figures 1 and 2 are diagrams prepared from the data in Tables 4 and 5 and show comparative results.

Table No. 6 shows percentage of voids in each of original mixtures, that passing a No. 16 sieve, that retained on a No. 16 and that retained on a No. 4. These data were secured with the idea of using them in making a theoretical mixture, but this was found impractical. They are given here simply as a matter of information.

In conclusion we would say that while not all the samples showed an increase in strength, notably sample 3, the test was not wholly without results. Samples Nos. 2, 4 and 5 showed the greatest increase. No. 2 showed an increase of 18 per cent on the 7 day test and 26 per cent on the 28 day test. Sample No. 4 showed an increase of 15 per cent on the 7 day test, 23 per cent for the 28 day test, and 18 per cent for the 3 month test. Sample No. 5 showed an increase of 33.8 per cent for 7 day test, 39 percent for the 28 day test, and 27 percent for the 3 month test.

While it is not advisable or practical to make mixtures of all sands, the cost of testing is not great and in some cases such tests may give flattering results.

In case such tests give good results, the question then arises whether it will be cheaper to obtain the desired increase in strength by making such a mixture as has been described or by increasing the amount of cement.

The following data were used in making these computations.

Cement $2 per barrel. As a barrel of cement contains about four cubic feet, a cubic yard of neat cement would cost $13.50.

Cost of sand at pit	.50 per cu. yd.
Cost of sand on work	1.00 per cu. yd.
Cost of sand sieving	.15 per cu. yd.

From table 1a take 35 per cent of size retained on a No. 16 sieve as amount to be added. Only 35 per cent of sieved sand is used the remainder is wasted. Then for every yard of sieved sand used, 1.86 yards are wasted, making the total cost of a yard of the sieved sand, which is to be added to the mixture, $1.85 at the pit.

From table No. 2 it is seen that the sieved sand is about 1-3 the mixture. This makes the cost of the mixture $1.05 per yard at the pit or $1.55 on the work.

From figures 1 and 2 take average gain in strength 25 per cent. By Baker's Masonry Construction, pg. 86 it appears that 1 cu. yd. of 1:3 mortar requires 0.90 cu. yd. of sand which will cost $0.55 per cu. yd. more, according to the discussion just given, if graded than if the original sand is used. Hence one cu. yd. of mortar made with the graded sand will cost about $.50 more than if made with the original ungraded sand, while by figures 1 and 2 it would be about 25 per cent stronger.

By Baker's Masonry Construction, Pg. 90 it appears that the same gain of 25 per cent in strength could be secured by using more cement, and that this would require a 1:2¼ mixture instead of 1:3. By the same authority Pg. 86 this would require about 0.6 of a barrel of cement more per cu. yd. than the 1:3 which would cost $1.20 more than the 1:3 mortar, or about $1.10 after allowing for a slight saving in the amount of sand.

Hence in the case assumed the increase in strength is secured at less than half the cost by grading the sand, as indicated in this thesis, in preference to using more cement.

In these computations it has been necessary to make many rough estimates of amounts and costs of materials, hence their value lies not in the accuracy of the results obtained, but in the method which shows how accurate results might be obtained from accurate data. They show that in many cases a great saving may be effected by making a practical mixture.

TABLE NO IA. ANALYSIS OF SAMPLES.
SHOWING PERCENTAGES RETAINED ON EACH SIEVE.

	No. 1	No. 2	No. 3	No. 4	No. 5
Retained on No. 2	1.0	1.0	28.1	1.0	7.2
Retained on No. 4	3.8.	4.7	45.8	4.0	13.0
Retained on No. 8	12.6	16.4	65.5	14.0	22.0
Retained on No. 10	16.5	24.0	70.7	20.0	25.6
Retained on No. 16	27.2	35.8	77.5	32.0	32.4
Retained on No. 20	35.3	44.4	83.0	41.0	39.0
Retained on No. 30	51.0	60.4	90.0	63.0	50.7
Retained on No. 40	68.2	77.0	96.4	75.0	6[illegible]
Retained on No. 50	75.5	81.0	97.5	91.5	78.0
Retained on No. 80	83.5	97.0	98.5	96.5	90.0
Retained on No.100	95.0	99.0	99.0	98.0	93.0
Retained on No.200	99.1	99.8	99.4	99.7	97.5

TALLE NO. Ib. ANALYSIS OF SAMPLES.

Sample	Voids By Displ'c't	Voids By Sp. Gr.	Ret. on No. 4	Ret. on No.16	Passed No. 16	Sp. Gr.
No. 1	35	31	3.7	27.2	72.8	2.65
No. 2	29	27	4.7	35.8	64.2	2.63
No. 3	32	27	45.8	77.5	22.5	2.68
No. 4	32	31	4.0	32.0	68.0	2.61
No. 5	31	29	13.0	32.4	67.6	2.60

Note:—Percentage voids for Std. Sand 34 by displ't and 32 by Sp. Gr.

TABLE NO. II. MIXTURES.

Sample	Amt. of Orig.	Amonnt Added Ret. on No. 16	Amonnt Added Pass No. 16	Resulting voids
No. 1	100 cc.	60 cc.	25 cc.	30
" 2	" "	50 "		27
" 3	" "			27
" 4	" "	60 "		29
" 5	" "	70 "		26.5

NOTE—Percentage voids determined by displacement.

TABLE NO. 3. TEST OF CEMENT USED.

Atlas Portland Cement. Neat Cement Briquettes.

No.	24 hr.	7 day	28 day	3 mo.
1	140	525	675	570
2	135	460	555	660
3	135	450	665	550
4	145	535	420	610
5	150	500	550	645
6	145	505	N. G.	
7	150	560	635	
8	155	610	750	
9	157	575	630	
10	163	505	650	
Avg.	148	523	615	607

1:3 Standard Sand,

No.	24 hr.	7 day	28 day	3 mo.
1		158	275	260
2		158	280	270
3		130	250	220
4		105	250	330
5		180	240	210
6		150	230	220
7		146	250	280
8		147	260	310
9		158	285	220
10		125	235	
Avg.		148	256	258

TABLE NO. IV. TESTS OF ORIGINAL SAND.

Sample No. I.

No.	7 Day	28Day	3 mo.
1	140	200	
2	158	190	
3	190	190	
4	140	220	
5	165	160	
6	170	180	
7	130	185	
8	163		
9	120		
10	130		
Avg.	151	189	

Sample No. II.

No.	7 Day	28Day	3 mo.
1	190	410	
2	235	310	
3	275	325	
4	230	350	
5	280	355	
6	220	375	
7	270	340	
8	240	290	
9	285		
10	245		
Avg.	247	344	

Sample No. III.

No.	7 Day	28Day	3 mo.
1	260	320	
2	290	305	
3	240	390	
4	270	375	
5	200	370	
6	275	290	
7	280	360	
8	265	360	
9	215	380	
10	240	370	
Avg.	254	352	

Sample No. IV.

No.	7 Day	28Day	3 mo.
1	165	190	220
2	190	200	270
3	195	200	260
4	175	215	215
5	190	245	245
6	145	220	250
7	155	140	245
8	140	145	225
9	195	275	
10	205	170	
Avg.	176	200	241

Sample No. V.

No.	7 Day	28Day	3 mo.
1	135	220	200
2	100	175	230
3	155	170	205
4	125	160	225
5	155	200	210
6	115	190	220
7	105	170	200
8	115	195	210
9	120	180	230
10	140	160	215
Avg.	127	182	215

TABLE NO V. TESTS OF MIXTURES.

Sample No. 1

No.	7 day	28 day	3 mo.
1	110	230	
2	110	210	
3	105	220	
4	120	200	
5	105	200	
6	115	175	
7	124	210	
8	110	180	
9	130	180	
10	120	220	
Avg.	114	202	

Sample No. 2

1	305	410	
2	270	420	
3	280	440	
4	235	405	
5	265	470	
6	315	435	
7	280	420	
8	260	470	
9	260		
10	355		
Avg.	293	434	

Sample No. 3

1	285	300	
2	300	340	
3	290	330	
4	270	310	
5	280	330	
6	220	370	
7	280	355	
8	295	335	
9	270	335	
10	230	300	
Avg.	272	330	

Sample No 4

No.	7 day	28 day	3 mo.
1	235	260	310
2	180	270	280
3	190	215	275
4	200	270	260
5	170	265	280
6	185	310	310
7	175	180	260
8	190	220	260
9	195	250	290
10	200	250	340
Avg.	192	247	286

Sample No. 5

1	185	225	320
2	170	305	225
3	180	295	320
4	190	305	260
5	155	250	300
6	125	205	300
7	175	185	210
8	175	210	245
9	170	310	280
10	180	240	
Avg.	170	253	273

TABLE NO. VI

Percentage of Voids in each size of sand before mixing.

Sample	Orig. Sand	Pass No. 16	Ret. No. 16	Ret. No. 4
No. 1	35	36	40	49
No. 2	29	34	37	50
No. 3	32	35	34	49
No. 4	32	34	35	44
No. 5	31	33	37	50

FIG. No. 1.

COMPARATIVE STRENGTH OF ORIGINAL SAND AND MIXTURES SHOWN GRAPHICALLY.

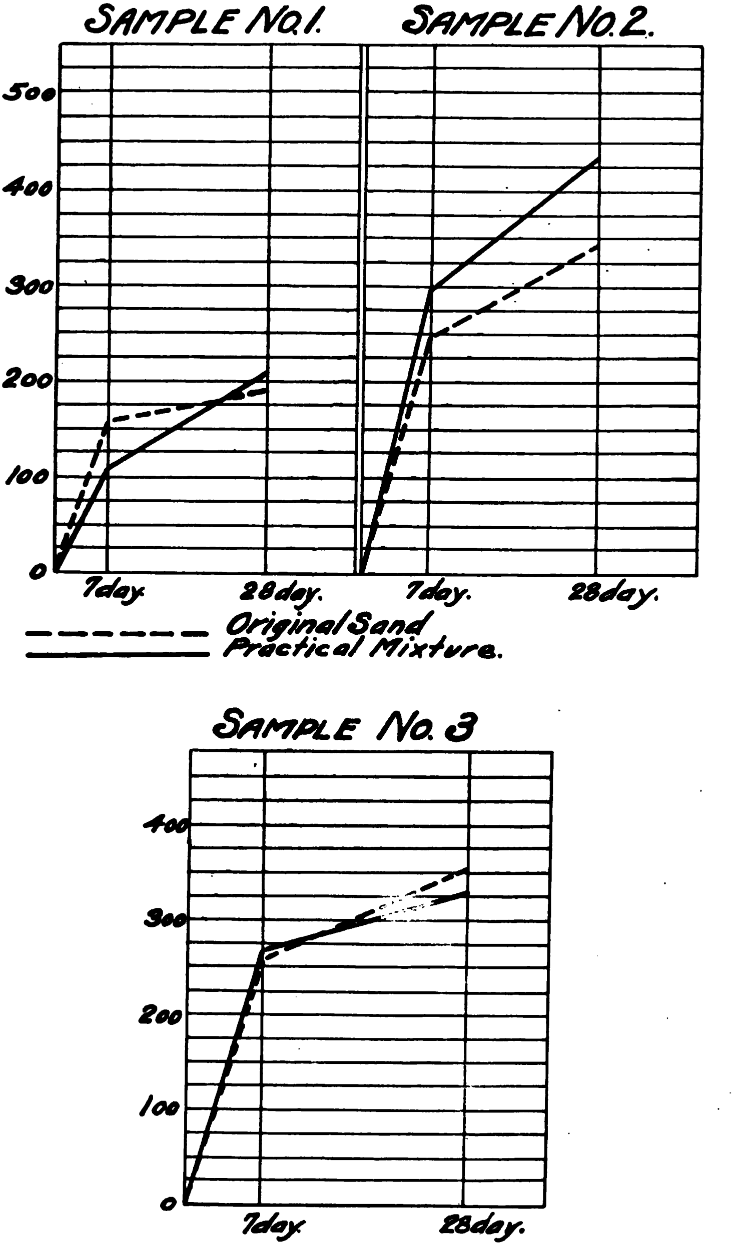
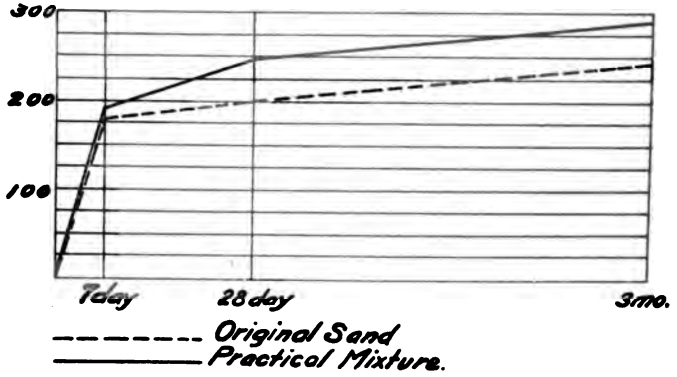


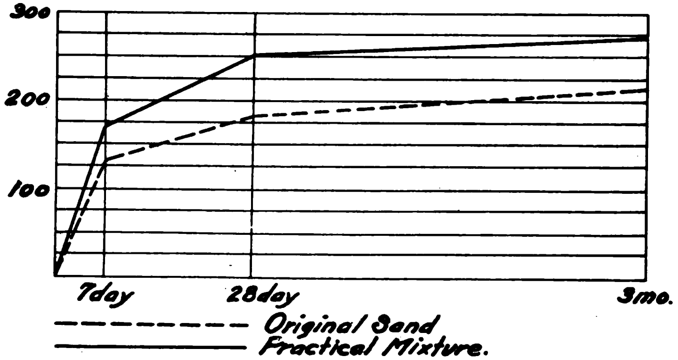
FIG. No. 2.

COMPARATIVE STRENGTH OF ORIGINAL SAND AND MIXTURES SHOWN GRAPHICALLY

SAMPLE No. 4.



SAMPLE No. 5.



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BULLETIN

VOL. IV No IV

The Determination of the Proper Proportions for Mortar and Concrete.......R. R. Strothers and Platt Wilson

The Effect of Clay and Reground Cement on the Permeability of Concrete and Cement Mortars.......O. L. Huffman and E. S. Fowler

Tests of Patent Compounds for Waterproofing Concrete....Royce Heath

An Investigation of the Change in Volume of Concrete During the Hardening Period.......C. W. Okey and W. H. Leckliter

The Coefficient of Roughness in Tile Drains.......Jno. F. Rightmire and Milford Chappel

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DECEMBER 1909

INTRODUCTORY

The five chapters comprising this bulletin are reports of thesis work done by members of the 1909 class in civil engineering at the Iowa State College. These papers were submitted in competition for the Miracle award. Mr. O. U. Miracle, of Minneapolis, Minnesota, a former student of the college, offers each year a prize of One Hundred dollars to be distributed among the three contestants whose papers are regarded by the judges as of the greatest practical value to the cement and concrete industry. Fifty dollars are awarded as the first, thirty dollars the second and twenty dollars as the third prize. The judges in this year's contest were, Mr. L. L. Bingham of Estherville, Iowa, Mr. Geo. P. Dieckmann of Mason City, Iowa and Mr. Ira A. Williams, of the Iowa State College, Ames, Iowa. The papers were ranked by the judges in the order of their arrangement in this bulletin, the first three only receiving prizes.

These contributions are suggestive and, it is believed, of sufficient importance to warrant their publication and distribution as a bulletin of the Engineering Experiment Station. Cement users and others interested along related lines will find in them many valuable concrete facts besides suggestive data for the solution of some of the problems that come to the practical worker in cement and concrete. A complete list of the publications of the Station is appended at the close of this bulletin, copies of which may be obtained by addressing the Director, A. Marston, Iowa State College, Ames, Iowa.

Ira A. Williams, Editor.

THE DETERMINATION OF THE PROPER PROPORTIONS FOR MORTAR AND CONCRETE.

BY R R. STROTHERS AND PLATT WILSON.

It is a well known fact that the densest concrete is the strongest and that the densest concrete is one in which all the voids in the stone are just filled with sand and in which all the voids in the sand are just filled with cement and with a very thin film of cement between each of the particles. It may seem at first thought that if 100 percent of the voids in the rock be filled with cement, we should have the proper proportions to make the most dense concrete; but such is not the case as has been shown by several experimenters. More than 100 percent of sand is required to fill the voids in the stone and more than 100 percent of cement is required to fill th voids in the sand; the explanation being that the grains of sand, getting between the pieces of rock, push them apart, and the cement, getting between the grains of sand, push them apart thus making the resulting mixture larger in volume than the original volume of crushed rock, by increasing the voids in the rock and the sand.

It is the object of this thesis to determine practically the proper proportions for a concrete in which all the voids are filled. This is to be done by making a mortar in which all of the voids in the sand are filled with neat cement and, second, by making a concrete in which all of the voids in the crushed rock are just filled with the properly proportioned cement mortar. Lastly, it is to be proved that such a concrete, being the densest mixture possible is stronger than all weaker mixtures and as strong, if not stronger than all richer mixtures.

Northwestern States Portland Cement was used in this work which, when tested, was found to conform to the specifications of the American Society for Testing Materials.

Cement Tests: Specific gravity, 3.16; fineness test, 23.9 percent retained on No. 200 sieve. 5.1 percent retained on No. 100 sieve.

Normal consistency; 23½ percent water required.

Constancy of volume; pats exposed in dry air 28 days, ok. Pats exposed in water 28 days, ok. Pats exposed to steam 5 hours, ok.

Time of setting; Initial set in 1¾ hours. Final set in 4½ hours.

Tensile test; Neat, after 24 hours average of briquets—259 lbs. per sq. in. Neat, after 7 days average of briquets—671.6 lbs. per sq. in. Neat, after 28 days average of briquets—780 lbs. per sq. in.

One cement to 3 sand; 7 day test, average of briquets—180 lbs. per sq. in. 28 day test, average of briquets—268 lbs. per sq. in.

The cement measured loose weighed 80 pounds per cubic foot. Neat cement measured loose was found to decrease 20 percent in volume when made into neat motrar, that is, 100 cubic yards of dry cement would make only 80 cubic yards of neat mortar.

Sand.—The sand used was common bank sand obtained from a local sand bank. A sizing test gave:

11.1 per cent retained on the No. 10 sieve.
32.05 per cent retained on the No. 20 sieve.
22.33 per cent retained on the No. 30 sieve.
16.93 per cent retained on the No. 40 sieve.
9.52 per cent retained on the No. 50 sieve.

The sand was found to contain 5.35 per cent of clay, the remaining 94.65 being clean quartz.

The voids in the sand were determined in the following manner.

A 100 c.c. flask was filled with hot sand, tapping the flask to insure its being compactly filled. The sand was then weighed. This weight of the sand was divided by the specific gravity of quartz (2.65) and the result is the per cent of quartz or sand. The difference between this and 100 gives the per cent of voids in the sand. The average of three trials was found to give 35.5 per cent voids, neither trial varying more than .12 per cent from the average.

The weight of the sand was 88 pounds per cubic foot measured loose. By actual test the rammed sand was found to increase 21.5 per cent when measured loose, or it would take 121.5 cubic yards of loose sand to make 100 cubic yards of rammed sand. The weight of the sand packed was 108 pounds per cubic foot.

In carrying out the tests a form 6 by 6 by 30 inches was used making a beam containing just 5-8 cubic foot of mortar.

The tests were made in the following manner. At the beginning enough sand was taken to just fill the form when rammed. Since there are 35.5 per cent voids in the sand, in .625 cubic feet of sand there are .2218 cubic feet of voids, of which, 80 per cent are to be filled with neat cement mortar, requiring .177 cubic feet of the latter. Since neat cement mortar is only 80 per cent of loose cement, it will take .2218 cubic feet of loose cement to fill 80 per cent of the voids. This .625 cubic feet of packed sand and .2218 cubic feet of loose cement are mixed with the proper amount of water into a mortar. The mortar is then placed in the form and is well rammed. It is found however that all of the mortar cannot be rammed into the form, showing that part of the neat cement has pushed the grains of sand apart.

After the 6 by 6 by 30 inch form has been filled, the remaining mortar is measured and the volume of the rammed mortar thus obtained. The volume of rammed mortar divided by the volume of rammed sand gives the volume of rammed mortar in terms of the rammed sand.

In this case we have 1080 cubic inches of rammed sand making 1215 cubic inches of rammed mortar, which makes the rammed mortar 112.5 per cent of the rammed sand. In other words, 100 cubic yards of rammed sand make 112.5 cubic yards of rammed mortar.

Out of 100 cubic yards of rammed sand 35.5 are voids, 64.5 cubic yards being solid sand. If the neat mortar used in the mixture is 80 per cent of the 35.5 cubic yards of voids, we have 28.4 of neat cement plus 64.5 of solid sand, which gives 92.9 cubic yards of solid material in the mixture, measuring when mixed 112.5 cubic yards, therefore, there must be 19.6 cubic yards in the mixture which are voids, surplus water, etc. 19.6 is 17.41 per cent of 112.5 cubic yards and is the percentage of voids in the rammed mortar.

Table No. 1 shows the results obtained as above from several mixtures. By using enough cement to fill 80 per cent of the voids in the sand a mixture is made having 17 per cent of voids.

TABLE NO. 1. MORTAR

Neat Mortar in terms of voids in the sand	Cubic feet of packed sand	Cubic feet of loose cement	Proportions	Weight of cement	Weight of 10.5% Water	Cubic inches of mortar	Excess of mortar in cubic inches	Volume of rammed mortar in terms of volume of sand	Voids in rammed mortar
80	.625	.2218	1:2.8	17.75 lbs.	8.95 lbs.	1215	135	112.5	17.41%
80	"	"	"	"	"	1216	126	111.7	16.83 "
100	"	2773	1:2.74	22.18 lbs.	9.42 lbs.	1252	172.8	116.0	13.80 "
100	"	"	"	"	"	1249.2	169.2	115.7	13.57 "
120	"	.3333	1:2.3	26.60 lbs.	9.88 lbs.	1296	216	120.0	10.75 "
120	"	"	"	"	"	1287	207	119.2	11.50 "
130	"	.3610	1:2.1	28.80 lbs.	10.25 lbs.	1314	234	121.6	9.00 "
130	"	"	"	"	"	1308	228	121.1	8.40 "
140	"	.3880	1:2	31.04 lbs.	10.34 lbs.	1342.8	262.8	124.3	8.10 "
140	"	"	"	"	"	1342.8	262.8	124.3	8.10 "
160	"	.4440	1:1.7	35.52 lbs.	10.82 lbs.	1404	324	130.0	6.70 "
160	"	"	"	"	"	1422	342	131.6	7.50 "
180	"	.4980	1:1.3	39.85 lbs.	11.27 lbs.	1494	414	138.3	7.20 "
180	"	"	"	"	"	1487	404	137.7	6.80 "

These figures indicate that, after enough cement to fill 140 per cent of the voids in the sand is used, further additions are useless, as the decrease in the voids in the mixture from that point on is very slight, showing practically that a mortar cannot be made in which there are no voids.

By using enough cement to fill 140 per cent of the voids in the sand we have practically as dense a mortar as can be made and the results from the cross bending and crushing tests given in Table No. 3 show that this mixture gives a strength as high as is reached by richer mixtures. This mixture was therefore decided upon to use in making the concrete, its proportions, with the sand and cement used, measured loose by volume being, 1 cement to 2 of sand.

Stone.—The stone used in this test was crushed limestone which had passed a ¾ inch ring and was retained on a No. 8 sieve. The voids in the broken stone were determined by obtaining the specific gravity of the stone; the results of several tests showing a specific gravity of 2.48. The weight of ⅝ cubic feet of tamped broken stone was found to be 59.97 pounds while that of ⅝ cubic feet of water is 39.075 pounds. 39.075 x 2.48 equals 96.844 pounds, which is the weight of a solid stone containing ⅝ cubic feet. The weight of ⅝ cubic feet of crushed stone, 59.97 pounds, divided by the weight of a solid stone ,96.844, gives 61.92 per cent, which is the proportion of the ⅝ cubic feet of broken stone that is solid material, leaving 38.08 per cent of voids. The ⅝ cubic feet of rammed broken stone measured 1260 cubic feet loose and weighed 59 pounds and 15.5 ounces.

Mortar.—The mortar of the proportions 1 cement to 2 of sand shows an increases, in the final mixture of 124.3 per cent of the volume of rammed sand used. Taking 100 cubic yards as a unit, it has been shown that 100 cubic yards of rammed sand is equal to 121.5 cubic yards of loose sand; also, 100 cubic yards of rammed sand will make 124.3 cubic yards of cement mortar of the 1 to 2 proportions. Therefore the volume of loose sand used in the 1 to 2 mortar is 97.7 per cent of the volume of the final mixture.

In the concrete tests the same forms were used as before

TABLE NO. 2. CONCRETE

1080 cubic inches rammed stone — Voids in Stone—38.08%

Per cent of voids in rammed broken stone filled with mortar	Cubic feet of mortar	Cubic feet of sand	Cubic feet of cement	Weight of water	Proportions	Cubic inches of concrete	Volume of rammed conerete in terms of volume of rammed stone	Excess of concrete in cubic inches	Voids in concrete
80	.19	.1856	.0928	8 lbs. 5 ozs.	1:2:7.85	1125	45	104.2%	12.3
						1125	45	104.2	12.3
100	.2375	.2320	.1160	8 lbs. 14.4 ozs.	1:2:6.22	1161	81	107.6	7.06
						1179	99	109.2	8.42
120	.2856	.2780	.1390	8 lbs. 14.4 ozs.	12:5.24	1220.4	140.4	113.0	4.78
						1195.2	115.2	112.6	4.53
140	.3325	.3250	.1625	10 lbs. 6 ozs.	1:2:4.48	1300	220	120.4	4.3
						1318	218	120.2	4.16
160	.3800	.3710	.1855	12 lbs. 00 ozs.	1:2:3.93	1484.4	304.4	128.1	4.14
						1383.5	303.5	128.1	4.09

TABLE NO. 3 MORTAR

Percentage of voids in the sand that are filled with neat mortar.	Compression.					Cross Bending.			
	NOTE:—Size of test blocks—6 inches x 6 inches x 6 inches—Angle of shear, 45 degrees; Age, 60 days.					NOTE:—Size of beams 6 inches x 6 inches x 30 inches. Distance between supports, 27 inches. Age, 60 days.			
	Spalling point.	Average.	Ultimate strength.	Average.	Strength per square inch.	Breaking load.	Average.	Bending moment.	Stress in outer fiber.
80	54000	67500	83200	85700	2380	2330	1990	13440	392
80	76000		88200			1650			
100	73000	74000	84600	85050	2360	1920	1855	12450	348
100	75000		85500			1790			
120	90000	97500	No Failure	110000+	3055	2400	2210	14920	414
120	105000		at			2020			
130	100000	86500	110000+			2160	2190	14728	409
130	73000					2220			
140	95000	97500				3400	2660	17950	499
140	100000					1920			
160	87000	88500				2690	2685	18120	504
160	90000					2680			
180	107000	105000				2280	2495	16840	468
180	103000					2718			

Taking the first test in Table No. 3 as an example; 1260 cubic inches equals .729 cubic feet of loose rock, weighing 59 pounds 15.5 ounces. 1080 cubic inches or .625 cubic feet when rammed will just fill the form. Since 38.08 per cent of this amount is voids, we have 2375 cubic feet of voids in .625 cubic feet of rammed crushed rock. Since we wish to fill 80 per cent of these voids, 19 cubic feet of mortar will be required. For a 1 to 2 mixture 97.7 per cent of .19 cubic feet or .18563 cubic feet of sand will be required and .0928 cubic feet of cement measured loose. The .0928 cubic feet of cement, .18563 cubic feet of sand and .729 cubic feet crushed rock are then mixed using just enough water to give the proper consistency. As much of the resulting concrete as is possible is then rammed into the form and the surplus carefully measured. The volume of the rammed concrete is thus obtaind. In this case it was found to be 1125 cubic inches which is 104.2 per cent of the rammed crushed rock. Taking 100 cubic yards as a unit, 100 cubic yards of rammed crushed rock will make 104.2 per cent rammed concrete of these proportions. In the 100 cubic yards of broken stone 38.08 cubic yards are voids. Since enough mortar is to be used to fill 80 per cent of the voids 30.464 cubic yards of mortar will be required. There are 61.92 cubic yards of solid rock which makes 91.38 cubic yards of solid material in the 104.2 cubic yards of concrete. There are then 12.82 cubic yards of voids in the concrete, showing it to be 12.3 per cent voids.

Table No. 2 shows a decrease of 4.56 in the percentage of voids for the 100 per cent mixture compared to the 80 per cent mixture, 3.09 for the 120 over the 100, .42 for the 140 over the 120, and .09 for the 160 over the 140. It is thus seen that after enough cement mortar to fill 120 per cent of the voids is used any further addition to cement is practically wasted as the decrease in the percentage of voids is very slight.

By an examination of Table No. 4 we find that the 120 per cent mixture shows higher results than all weaker mixtures and as strong as the richer mixtures.

TABLE NO. 4—CONCRETE

Percentage of voids in rammed stone filled with mortar	Compression					Cross Bending			
	Note—Size of test blocks: 6x6x6 inch cubes Angle of shear: 45 degrees Age: 60 days					Note—Size of beams: 6x6x36 inches Distance between supports: 27 inches Age: 60 days			
	Spalling point	Average	Ultimate strength	Average	Strength per square inch	Breaking load	Average	Bending moment	Stress in outer fiber
80	62000	61000	67200	66100	1836	1240	1610	10870	302
	60000		65400			1980			
100	74000	67500	78100	71350	1988	1860	1505	10160	289
	61000		64600			2150			
120	78000	74000	100000	91500	2540	2900	2963	20000	555
	70000		83000			3030			
140	77000	70000	93500	92250	2562	2700	2580	19250	635
	63000		91000			3000			
160	88000	76000	92500	90250	2507	2300	2620	17700	492
	64000		88000			2440			

When a concrete is made in which 120 per cent of the voids in the stone are filled with a cement mortar in which 140 per cent of the voids in the sand are filled with cement, we have a mixture which is practically the densest it is possible to make under ordinary conditions and one which, when tested, shows results stronger than all leaner mixtures and as strong as all richer mixtures. This then is the best mixture so far as economy and strength are concerned. The densest mixture is also the strongest.

THE EFFECT OF CLAY AND REGROUND CEMENT ON THE PERMEABILITY OF CONCRETE AND CEMENT MORTARS

BY O. L. HUFFMAN AND E. S. FOWLER.

In the early development of our country the great abundance of timber furnished a ready building material and one applicable to almost all kinds of construction; but with the increase in population, especially in the cities, wooden buildings were soon necessarily replaced by more permanent structures and of a material which would render them safer from fire. Stone, brick and steel were used for some time almost exclusively but their cost, which made them prohibitive for the man of ordinary means, led to the gradual substitution of concrete in their place, until at the present time, it is being used more and more extensively each year as a material of construction.

This has been partly due to the decrease in our lumber supply but more especially to the tendency toward more permanent structures. With concrete, as with other new building materials, many failures, due to carelessness and ignorance of the material, have greatly retarded its more general application. Its use in the presence of water has been seriously retarded on account of its permeability. This latter property has been the cause of considerable investigation of late in an attempt to discover a remedy.

The nature of the work done for this thesis was to investigate the value of reground cement and clay as waterproofing materials in cement mortars and concrete.

Materials Used.—Northwestern States Portland Cement was used. Tested in the laboratory it was found to conform to the specifications of the American Society for Testing Materials.

Reground Cement:—The reground cement was made from the ordinary factory product by grinding it for fourteen hours in a ball mill, in the cement laboratory. About ten pounds were put in each cylinder at a grinding. The average fineness of the three grindings made was 98.5 per cent passing a No. 200 sieve.

Sand:—The sand used was obtained from a pit near the college, all being used which passed a No. 8 sieve. The sand analysis was as follows::

Retained on No. 8 sieve 27.75 per cent.
Retained on No. 20 sieve 22.66 per cent.
Retained on No. 30 sieve 12.35 per cent.
Retained on No. 50 sieve 27.75 per cent.
Retained on No. 100 sieve 7.20 per cent.
Retained on No. 200 sieve 0.72 per cent.
Passed No. 200 sieve 1.44 per cent.
Total ....99.87 per cent.

The voids were 36 per cent. The amount of clay was 3.5 per cent.

Clay.—The clay used was of good quality, free from sand. It was dried at a low heat for twenty-four hours and was then ground until 98 per cent passed a No. 200 sieve.

Stone.—The stone used was a good quality Iowa limestone, broken to pass a 1.5 inch ring. All dust passing a No 8 sieve was removed.

The mixtures used were as follows:

1 common cement to 2 sand.
1 reground cement to 2 sand.
.9 common cement to .1 clay to 1 sand.
.85 common cement to .15 clay to 2 sand.
1 common cement to 3 sand.
1 reground cement to 3 sand.
.9 common cement to .1 clay to 3 sand.
.5 common cement to .15 clay to 3 sand.
1 common cement to 2 sand and 4 broken stone.
1 reground cement to 2 sand and 4 broken stone.
.9 common cement and .1 clay to 2 sand and 4 broken stone.
.85 common cement and .15 clay to 2 sand and 4 broken stone.
1 common cement to 3 sand and 5 broken stone.
1 reground cement to 3 sand and 5 broken stone.

In the mixtures of clay and common cement, the clay replaced part of the cement.

Mixing. Proportioning was by weight. The mixing was done by hand; as nearly as possible imitating actual operations as practiced in the field. In the case of the clay it was first mixed with the cement and then with the sand and broken stone. In filling the forms the material was well tamped to make as dense a mixture as possible.

Method of Testing. No standard testing apparatus had been adopted by the college for testing specimens under pressure and it was, therefore, necessary to design one that would be adequate for extensive tests. The apparatus used was suggested by Mr. Royce Heath and after a few modifications, adopted for our work.

The test specimens were made by filling forms of corrugated galvanized iron, each in shape the frustrum of a cone; 4 inches in diameter at the larger end and 3 1/3 inches at the other. These were filled to within 1 inch of the larger end.

Two sizes of test pieces were used—6 and 3 inches thick—and three specimens of each size for each mixture were made, making 84 specimens all told.

The tests were divided into three series with regard to age; viz: thirty, forty and sixty day specimens. One 6 inch and one 3 inch specimen were used for each test. The specimens were all tested under pressure by being clamped on to a cast-iron head, (as shown in plan of apparatus) through which the water was conducted by a 1/2 inch pipe, connected to an inlet pipe from the college pressure mains. The connection between the test piece and the cast-iron head was made water-tight by means of a rubber gasket placed in a groove in the head.

The pressure was read by a pressure-gauge attached to the inlet pipe; the maximum pressure from the mains being about 60 pounds per square inch. The water was started at 30 pounds per square inch and at the end of six hours was increased to 40 pounds. After 16 hours at 40 it was increased to 60 pounds and this pressure was maintained from 7 to 22 hours in the initial tests.

Nearly all of the specimens were broken open at the end of tests and the penetration of water, or moisture line, observed.

For the tension test briquettes of standard size were made and tested in a Fairbanks Standard Testing machine. The briquettes were made along with the specimens for the pressure test, and from the same material. They were all stored in moist air for twenty-four hours and then put under water. The object of this test was to compare the strength of the different mixtures.

In the absorption tests the specimens were cubes 3x3x5 inches made from the same mixtures as the pressure specimens. They were allowed to set in dry air for one week, then weighed and stored under water for ninety days. They were then removed, wiped dry, and reweighed and the percentage of absorption determined.

In the pressure test no measurement of the water passing the material in the 30 day test was made, but in the forty and sixty day specimens and all special tests, the water was caught in pans and measured. In the initial tests the rate of drip per hour was noted while in the special tests the total amount was measured. The tops of all specimens in the pressure test were kept filled with water to prevent any tendency of the material to shrink away from the forms. In fastening the specimens on the testing apparatus the water was allowed to force out the air in the top of the form before the final clamping to the rubber gasket, as it was noticed that otherwise the air confined between the pressure face of the specimen and the cast-iron head would be forced out through the concrete.

Definitions of Terms. The word "damp" is meant to signify the first stage from dryness,—a slight discoloration. The word "moist" is intended to signify the second stage from dryness, when, by passing the fingers over the outer surface moisture will be retained. The term "wet" is meant to signify the third stage from dryness and includes all not damp or moist. The word "saturated" used in describing broken specimuns means that water oozed out of the material when broken. The (*) signifies that the specimen was saved for a special test.

Special Tests. The best specimens of the initial test were reserved for a special test. In these tests the pressure was applied at once at 60 pounds per square inch and left on continually until the specimen was removed.

INITIAL TESTS

MORTARS

THIRTY DAY SPECIMENS.

1:2; *Common Cement*:

No. 8—3 inches thick.

Face was dry throughout entire test.

*No. 12—6 inches thick

Wet spots appeared on face in 10 minutes after pressure was applied.

30 pounds pressure for 6 hours. Face drying.

40 pounds pressure for 15 hours. Face moist all over.

60 pounds pressure for 10 hours. No change.

1:2, 10 *per cent Clay*:

No 30—3 inches thick

30 pounds pressure for 6 hours. Face half wet. Dripping at rate 100 per minute.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. Dripping changed to a small stream. Clay being washed out.

No. 25—6 inches thick

30 pounds pressure for 6 hours. Face ⅓ wet.

40 pounds pressure for 15 hours, Face ⅔ wet.

60 pounds pressure for 10 hours. Wet area slightly increased.

Broken and found wet in 3 inches on pressure face.

1:2, 15 *per cent Clay*:

*No. 37—3 inches thick

30 pounds pressure for 6 hours. Wet area ⅔ face.

40 pounds pressure for 15 hours. Ditto.

60 pounds pressure for 10 hours. Ditto.

No. 31—thickness 6 inches

30 pounds pressure for 6 hours. Face wet all over.

40 pounds pressure for 15 hours. Face moist all over.

60 pounds pressure for 10 hours. Face wet all over.

Broken—saturated to within 2 inches of outer face.

1:2, *Reground Cement*:

*No. 15—3 inches thick

30 pounds pressure for 6 hours Face absolutely dry.

40 pounds pressure for 15 hours. Ditto.

60 pounds pressure for 10 hours Ditto.

*No. 17—6 inches thick

30 pounds pressure for 6 hours. Face absolutely dry.

40 pounds pressure for 15 hours. Ditto.

60 pounds pressure for 10 hours. Ditto.

1:3, *Common Cement*:

No. 6—3 inches thick

30 pounds pressure for 6 hours. Face moist and dripping.

40 pounds pressure for 15 hours. Face wet all over.

60 pounds pressure for 10 hours. Ditto.

Broken—found saturated throughout.

No. 2—6 inches thick

30 pounds pressure for 6 hours. Edge wet. Center dry.

40 pounds pressure for 15 hours. Face nearly all wet.

60 pounds pressure for 10 hours. Face wet all over.

Broken—found entirely saturated throughout.

1:3, 10 *per cent Clay*:

No. 42—3 inches thick

30 pounds pressure for 6 hours. Face wet.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. Ditto.

*No. 39—6 inches thick

30 pounds pressure for 6 hours. Edge damp.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

Note:—No. 42. Broken—saturated throughout.

1:3, 15 *per cent Clay*:

No. 47—3 inches thick

30 pounds pressure for 6 hours. Face wet. 3 square inches.

40 pounds pressure for 15 hours. Face wet all over.

60 pounds pressure for 10 hours. Ditto—leaking badly.

No. 43—6 inches thick

30 pounds pressure for 6 hours. Edge damp.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. Edge moist. Dry area 2 square inches.

Broken—found saturated throughout.

1:3, *Reground Cement*:

No. 22—3 inches thick

Face became wet at once around edge and spread.

30 pounds pressure for 6 hours. Dry area 3 square inches.

40 pounds pressure for 15 hours. Dry area 2 square inches.

60 pounds pressure for 10 hours. Dry area 2 square inches.

Appearances indicated that water was coming thorugh next to form. Left on 31 hours longer and leaking gradually ceased. Broken—found wet in 1¼ inches on pressure side. Rest dry.

*No. 20—6 inches thick

30 pounds pressure for 6 hours. Face absolutely dry.

40 pounds pressure for 15 hours. Ditto.

60 pounds pressure for 10 hours. Ditto.

FORTY DAY SPECIMENS.

1:2, *Common Cement*:

No. 9—3 inches thick

Leaked at once around edge.

30 pounds pressure for 6 hours. Face wet all over.

40 pounds pressure for 15 hours. Leaking 12,000 cubic centimeters per hour.

Broken—found wet in 2 inches on pressure face. The leak was due to improper bond.

No. 11—6 inches thick

Wet around edges, dry in center.

30 pounds pressure for 6 hours. Leaking 4 cubic centimeters per hour.

40 pounds pressure for 16 hours. Leaking 5 cubic centimeters per hour.

60 pounds pressure for 22 hours. Leaking 8 cubic centimeters per hour.

Face wet all over.

Broken—found entirely saturated.

1:2, 10 *per cent Clay*:

*No. 28—3 inches thick

30 pounds pressure for 6 hours. Leaking 24 cubic centimeters per hour.

40 pounds pressure for 16 hours. Leaking 8 cubic centimeters per hour.

60 pounds pressure for 7 hours. Leaking 29 cubic centimeters per hour.

60 pounds pressure for 15 hours. Leaking 4 cubic centimeters per hour.

*No. 27—6 inches thick

30 pounds pressure for 6 hours. Dry area in center 3 square inches. Leaking 33 cubic centimeters per hour.

40 pounds pressure for 16 hours. Leaking 11 cubic centimeters per hour.

60 pounds pressure for 7 hours. Leaking 37 cubic centimeters per hour.

60 pounds pressure for 16 hours. Leaking 8 cubic centimeters per hour.

1:2, 15 *per cent Clay*:

*No. 34—3 inches thick

30 pounds pressure for 6 hours. Dry area 4 square inches. Leaking 250 cubic centimeters per hour.

40 pounds pressure for 16 hours. Leaking 316 per hour.

60 pounds pressure for 7 hours. Leaking 16 per hour.

60 pounds pressure for 15 hours. Leaking 39 per hour. Dry area decreased to 2 square inches.

*No. 33—6 inches thick

30 pounds pressure for 6 hours. Leaking 18 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 7 cubic centimeters per hour. Edge wet.

60 pounds pressure for 16 hours. Leaking 29 cubic centimeters per hour. Edge wet.

60 pounds pressure for 15 hours. Leaking 12 cubic centimeters per hour. Face moist.

1:2, *Reground Cement*:

*No. 16—3 inches thick

30 pounds pressure for 6 hours. Leaking 15 cubic centimeters per hour. Dry area 2 square inches.

40 pounds pressure for 16 hours. Leaking 5 cubic centimeters per hour. Face dry. Edge moist.

60 pounds pressure for 7 hours. Leaking 29 cubic centimeters per hour. Face moist.

60 pounds pressure for 15 hours. Leaking 20 cubic centimeters per hour.

*No. 13—6 inches thick

30 pounds pressure for 6 hours. Leaking 7 cubic centimeters per hour. No change.

40 pounds pressure for 16 hours. Leaking 3 cubic centimeters per hour. Edge wet.

60 pounds pressure for 17 hours. Leaking 9 cubic centimeters per hour. Edge wet.

60 pounds pressure for 15 hours. Leaking 1 cubic centimeters per hour. Edge wet.

1:3, *Common Cement*:

No. 5—3 inches thick

30 pounds pressure for 6 hours. Leaking 4 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 5 cubic centimeters per hour. Face moist.

60 pounds pressure for 7 hours. Leaking 5 cubic centimeters per hour. Face moist.

60 pounds pressure for 15 hours. Leaking 5 cubic centimeters per hour. Face moist.

Broken—found entirely saturated throughout.

No. 3—6 inches thick

30 pounds pressure for 6 hours. Leaking 5 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 2 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 3 cubic centimeters per hour. Face wet.

60 pounds pressure for 15 hours. Leaking 3 cubic centimeters per hour. Face wet

Broken—found entirely saturated throughout.

1:3, 10 *per cent Clay*:

No. 41—3 inches thick

30 pounds pressure for 6 hours. Leaking 25 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 10 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 183 cubic centimeters per hour. Face wet

60 pounds pressure for 16 hours. Leaking 183 cubic centimeters per hour. Face wet

Broken—found completely saturated.

No. 38—6 inches thick

30 pounds pressure for 6 hours. Leaking 25 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 15 cubic centimeters per hour. Edge moist

60 pounds pressure for 7 hours. Leaking 22 cubic centimeters per hour. Edge moist

60 pounds pressure for 16 hours. Leaking 23 cubic centimeters per hour. Edge moist

Broken—found saturated to within 1½ inches outer face.

1:3, 15 *per cent Clay*:

No. 48—3 inches thick

30 pounds pressure for 6 hours. Leaking 433 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 410 cubic centimeters per hour. Face wet

60 pounds pressure for 7 hours. Leaking 430 cubic centimeters per hour. Face wet

60 pounds pressure for 15 hours. Leaking 430 cubic centimeters per hour. Face wet.

Broken—showed water coming through face.

*No. 45—6 inches thick

30 pounds pressure for 6 hours. Leaking 10 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 5 cubic centimeters per hour. Face moist.

60 pounds pressure for 7 hours. Leaking 19 cubic centimeters per hour. Face moist

60 pounds pressure for 15 hours. Leaking 5 cubic centimeters per hour. Face moist.

1:3, *Reground Cement*:

No. 24—3 inches thick

30 pounds pressure for 6 hours. Leaking 346 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 337 cubic centimeters per hour. Edge wet.

60 pounds pressure for 7 hours. Leaking 430 cubic centimeters per hour. Edge wet.

60 pounds pressure for 15 hours. No change.

Broken—found entirely dry in interior. Wet in on pressure face ½ inch. Water came through next to form.

*No. 21—6 inches thick

30 pounds pressure for 6 hours. Leaking 5 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 12 cubic centimeters. Edge dryer.

60 pounds pressure for 7 hours. Leaking 9 cubic centimeters. Nearly dry.

60 pounds pressure for 15 hours. Leaking 3 cubic centimeters.

SIXTY DAY SPECIMENS.

1:2, *Common Cement*:

*No. 7—3 inches thick

30 pounds pressure for 6 hours. Leaking 30 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 1 cubic centimeter per hour. Edge moist.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Edge moist.

60 pounds pressure for 15 hours. Leaking 3 cubic centimeters per hour. Edge moist.

*No. 10—6 inches thick

30 pounds pressure for 6 hours. Leaking. Edge wet.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Edge moist.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Face damp.

60 pounds pressure for 15 hours. Leaking 1 cubic centimeter per hour. Face damp.

1:2, 10 *per cent Clay*:

No. 29—3 inches thick

30 pounds pressure for 6 hours. Leaking 900 cubic

centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 2820 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 4200 cubic centimeters per hour. Face damp.

60 pounds pressure for 15 hours. Leaking 6000 cubic centimeters per hour. Face damp.

Broken—wet within 1 inch of outer face.

*No. 26—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 1 cubic centimeters per hour. Edge moist.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Edge moist.

60 pounds pressure for 15 hours. Leaking 14 cubic centimeters per hour. Edge moist.

1:2, 15 *per cent Clay*:

*No. 36—3 inches thick

30 pounds pressure for 6 hours. Leaking 19 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 12 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 12 cubic centimeters per hour. Face damp.

60 pounds pressure for 15 hours. Leaking 12 cubic centimeters per hour. Face damp.

No. 32—6 inches thick

30 pounds pressure for 6 hours. Leaking 26 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 26 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 100 cubic centimeters per hour. Face wet.

Broken—wet in 2 inches on pressure face.

1:2, *Reground Cement*:

*No. 18—3 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hours. Edge damp.

40 pounds pressure for 16 hours. Leaking 2 cubic centimeters per hour. Edge damp.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Edge damp.

60 pounds pressure for 15 hours. Leaking 2 cubic centimeters per hour. Edge damp.

*No. 14—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hours. Edge damp.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hours. Edge damp.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hours. Edge damp.

60 pounds pressure for 15 hours. Leaking 5 cubic centimeters per hours. Edge damp.

1:3, *Common Cement*:

No. 4—3 inches thick

30 pounds pressure for 6 hours. Leaking 20 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 2 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Face damp.

60 pounds pressure for 15 hours. Leaking 3 cubic centimeters per hour. Face damp.

Broken—found completely saturated.

No. 1—6 inches thick

30 pounds pressure for 6 hours. Leaking 600 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 15 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 1 cubic centimeters per hour. Face damp.

60 pounds pressure for 15 hours. Leaking 20 cubic centimeters per hour. Face wet.

Broken—wet in 3 inches.

1:3, 10 *per cent Clay*:

*No. 40—3 inches thick

30 pounds pressure for 6 hours. Leaking 100 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 40 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 200 cubic centimeters per hour. Face wet.

60 pounds pressire for 15 hours. Leaking. No change.

*No. 37—6 inches thick

30 pounds pressure for 6 hours. Leaking 16 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 12 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 20 cubic centimeters per hour. Face wet.

1:3, 15 *per cent Clay*:

No. 46—3 inches thick

30 pounds pressure for 6 hours. Leaking 600 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking. Very wet.

60 pounds pressure for 7 hours. Leaking 2100 cubic centimeters per hours. Very wet.

Broken—found entirely saturated.

*No. 44—6 inches thick

30 pounds pressure for 6 hours. Leaking 10 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 12 cubic centimeters per hour. Edge wet.

60 pounds pressure for 7 hours. Leaking 12 cubic centimeters per hour. Edge wet.

1:3, *Reground Cement*:

*No. 23—3 inches thick

30 pounds pressure for 6 hours. Leaking 400 cubic centimeters per hour. Edge damp.

40 pounds pressure for 16 hours. Leaking 31 cubic centimeters per hour. Edge damp.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Edge damp.

60 pounds pressure for 15 hours. Leaking 4 cubic centimeters per hour. Edge damp.

*No. 19—6 inches thick

30 pounds pressure for 6 hours. Leaking 20 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 2 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Face damp.

60 pounds pressure for 15 hours. Leaking 3 cubic centimeters per hour. Face damp.

CONCRETES

THIRTY DAY SPECIMENS.

1:2:4, *Common Cement*: ...

No. 64—3 inches thick

30 pounds pressure for 6 hours. Face wet; water standing on face.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

No. 61—6 inches thick

30 pounds pressure for 6 hours. Face wet in spots in five minutes after pressure was applied.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

1:2:4, 10 *per cent Clay*:

No. 76—3 inches thick

30 pounds pressure for 6 hours. Face wet all over.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

Broken—found completely saturated.

No. 75—6 inches thick

30 pounds pressure for 6 hours. Face wet at once.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

This specimen was not broken for it was evident that water was coming through concrete.

1:2:4, 15 *per cent Clay*:

No. 84—3 inches thick

30 pounds pressure for 6 hours. Face wet.

40 pounds pressure for 15 hours. Face wet and dripping freely.

60 pounds pressure for 10 hours. No change.

Broken—found entirely saturated.

No. 81—6 inches thick

30 pounds pressure for 6 hours. Face wet; dripping freely.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

1:2:4, *Reground Cement*:

*No. 52—3 inches thick

30 pounds pressure for 6 hours. Absolutely dry.

40 pounds pressure for 15 hours. Absolutely dry.

60 pounds pressure for 10 hours. Absolutely dry.

*No. 50—6 inches thick

30 pounds pressure for 6 hours. Absolutely dry.

40 pounds pressure for 15 hours. Absolutely dry.

60 pounds pressure for 10 hours. Absolutely dry.

1:3:5, *Common Cement*:

No. 71—3 inches thick

30 pounds pressure for 6 hours. Face wet.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

Note:—These two were not broken because it was evident that the water was coming through the concrete.

No. 69—6 inches thick

30 pounds pressure for 6 hours. Face wet all over.

40 pounds pressure for 15 hours. No change.

60 pounds pressure for 10 hours. No change.

1:3:5, *Reground Cement*:

No. 60—3 inches thick

30 pounds pressure for 6 hours. Edge damp.

40 pounds pressure for 15 hours. Face dry.

60 pounds pressure for 10 hours. No change.

Broken—wet in 1 inch on pressure face. Rest dry.

No. 57—6 inches thick

30 pounds pressure for 6 hours. Edge damp.

40 pounds pressure for 15 hours. Damp area decreased.
60 pounds pressure for 10 hours. No change.

FORTY DAY SPECIMENS.

1 :2 :4, *Common Cement*:

No. 65—3 inches thick

30 pounds pressure for 6 hours. Leaking 18 cubic centimeters per hour. Face wet.

40 pounds pressure for 15 hours. Leaking 68 cubic centimeters per hour. Face wet.

60 pounds pressure for 10 hours. No reading as leak was too great.

Broken—interior found wet in spots.

No. 62—6 inches thick

30 pounds pressure for 6 hours. Face damp.
40 pounds pressure for 16 hours. No change.
60 pounds pressure for 22 hours. No change.

1 :2 :4, 10 *per cent Clay*:

No. 78—3 inches thick

30 pounds pressure for 6 hours. Leaking 19 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 5 cubic centimeters per hour. Face wet.

60 pounds pressure for 22 hours. Leaking 2 cubic centimeters per hour. Face wet.

Broken—wet in 2 inches on pressure side and moist to outer face.

No. 74—6 inches thick

30 pounds pressure for 6 hours. Leaking 225 cubic centimeters per hour. Very wet.

40 pounds pressure for 16 hours. Leaking 64 cubic centimeters per hour. Very wet.

60 pounds pressure for 22 hours. Leaking 19 cubic centimeters per hour. Very wet.

Broken—saturated to within 2½ inches outer face.

1 :2 :4, 15 *per cent Clay*:

No. 82—3 inches thick

30 pounds pressure for 6 hours. Leaking 31 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 22 cubic centimeters per hour. Face wet.

60 pounds pressure for 22 hours. Leaking.

This specimen was taken off on account of excessive leaking. When broken it was found entirely wet through.

No. 79—6 inches thick

30 pounds pressure for 6 hours. Leaking 46 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 31 cubic centimeters per hour. Face wet.

60 pounds pressure for 22 hours. Leaking 39 cubic centimeters per hour. Face wet.

Broken—entirely saturated.

1:2:4, *Reground Cement*:

No. 53—3 inches thick

30 pounds pressure for 6 hours. Leaking 500 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 25 cubic centimeters per hour. Dryer.

60 pounds pressure for 22 hours. Leaking 00 cubic centimeters per hour. Edge damp.

Broken—absolutely dry except for edge.

*No. 49—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Edge moist.

60 pounds pressure for 22 hours. Leaking 2 cubic centimeters per hour. Edge moist.

1:3:5, *Common Cement*:

No. 72—3 inches thick

30 pounds pressure for 6 hours. Leaking 66 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 194 cubic centimeters per hour. Very wet.

60 pounds pressure for 22 hours. Leaking 120 cubic centimeters per hour. Very wet.

Broken—saturated throughout.

No. 67—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 19 cubic centimeters per hour. Face wet.

60 pounds pressure for 22 hours. Leaking 42 cubic centimeters per hour. Face wet.

Broken—saturated throughout.

1:3:5, *Reground Cement*:

*No. 58—3 inches thick

30 pounds pressure for 6 hours. Leaking 10 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 3 cubic centimeters per hour. Dryer.

60 pounds pressure for 22 hours. Leaking 1 cubic centimeters per hour. No change.

*No. 56—6 inches thick

30 pounds pressure for 6 hours. Leaking 5 cubic centimeters per hour. Edge moist.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Dryer.

60 pounds pressure for 22 hours. Leaking 00 cubic centimeters per hour. No change.

SIXTY DAY SPECIMENS.

1:2:4, *Common Cement*:

No. 66—3 inches thick

30 pounds pressure for 6 hours. Leaking 225 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 225 cubic centimeters per hour. Face wet.

60 pounds pressure for 22 hours. Leaking 330 cubic centimeters per hour. Face wet.

Broken—found saturated.

No. 63—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Face dry.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Face damp.

60 pounds pressure for 22 hours. Leaking 00 cubic centimeters per hour. Face wet.
Broken—saturated throughout.

1:2:4, 10 *per cent Clay*:

No. 77—3 inches thick
30 pounds pressure for 6 hours. Leaking 300 cubic centimeters per hour. Face wet.
40 pounds pressure for 16 hours. Leaking 200 cubic centimeters per hour. Face wet.
60 pounds pressure for 22 hours. Leaking 200 cubic centimeters per hour. Face wet.
Broken—saturated throughout.
No. 73—6 inches thick
30 pounds pressure for 6 hours. Leaking 18 cubic centimeters per hour. Face wet.
40 pounds pressure for 16 hours. Leaking 16 cubic centimeters per hour. Face wet.
60 pounds pressure for 22 hours. Leaking 10 cubic centimeters per hour. Face wet.
Broken—entirely saturated.

1:2:4, 15 *per cent Clay*:

No. 83—3 inches thick
30 pounds pressure for 6 hours. Leaking 10 cubic centimeters per hour. Face wet.
40 pounds pressure for 16 hours. Leaking 18 cubic centimeters per hour. Face wet.
60 pounds pressure for 7 hours. Leaking 20 cubic centimeters per hour. Face wet.
Broken—entirely saturated.
No. 80—6 inches thick
30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Face wet.
40 pounds pressure for 16 hours. Leaking 11 cubic centimeters per hour. Face wet.
60 pounds pressure for 7 hours. Leaking 18 cubic centimeters per hour. Face wet.
Broken—completely saturated.

1:2:4, *Reground Cement*:

*No. 54—3 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Edge damp.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Edge damp.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Edge damp.

60 pounds pressure for 15 hours. Leaking 7 cubic centimeters per hour. Edge damp.

*No. 51—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 00 cubic centimeters per hour. Face wet.

60 pounds pressure for 16 hours. Leaking 8 cubic centimeters per hour. Face wet.

1:3:5, *Common Cement*:

No. 70—3 inches thick

30 pounds pressure for 6 hours. Leaking 32 cubic centimeters per hour. Face wet.

40 pounds pressure for 16 hours. Leaking 20 cubic centimeters per hour. Face wet.

60 pounds pressure for 7 hours. Leaking 70 cubic

Water came through immediately after pressure was applied. Broken—entirely saturated.

No. 68—6 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Edge wet.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 13 cubic centimeters per hour. Face moist.

Broken—entirely saturated.

1:3:5, *Reground Cement*:

*No. 59—3 inches thick

30 pounds pressure for 6 hours. Leaking 00 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 2 cubic centimeters per hour. Face damp.

60 pounds pressure for 7 hours. Leaking 4 cubic centimeters per hour. Face moist.

*No. 55—6 inches thick

30 pounds pressure for 6 hours. Leaking 40 cubic centimeters per hour. Face damp.

40 pounds pressure for 16 hours. Leaking 00 cubic centimeters per hour. Face damp.

60 pounds prsesure for 7 hours. Leaking 00 cubic centimeters per hour. Dryer.

60 pounds pressure for 15 hours. Leaking 00 cubic centimeters per hour. No change.

No. 14—1 Regr'd Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
16	29	Edge damp
8	14	" "
20	27	" "
20	3	" "
9	00	" "
38	21	" "
33	00	" "
17	00	" "
161	94	3 inches thick 60 lbs. pressure
		Broken— Wet ⅛ inch on pressure face.

No. 17—1 Regr'd Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
6	00	Edge damp
16	00	" "
3	00	" "
17	00	" "
23	00	" "
22	00	" "
26	00	" "
116	00	6 inches thick 60 lbs. pressure
		Broken— Wet ¼ inch on pressure face.

No. 13—1 Regr'd Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	00	Edge damp
16	9	" moist
8	00	" "
17	00	" "
23	00	" "
26	00	" "
22	00	" "
116	0	6 inches thick 60 lbs. pressure
		Broken— Wet ⅛ inch on pressure face.

No. 16—1 Regr'd Cem. : 2 S.

No . Hrs.	No. ccs, passed	Remarks
4	44	Edge moist
16	105	" "
8	24	" "
17	42	" "
23	29	" "
26	14	" "
22	00	" "
116	262	3 inches thick 60 lbs. pressure
		Broken— Wet ¼ inch on pressure face.

No. 18—1 Regr'd Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
16	20	Edge damp
8	00	“ “
20	00	“ “
20	00	“ “
9	00	“ “
38	00	“ “
33	00	“ “
17	00	“ “
7	00	“ “
30	00	“ “
43	00	“ “
42	00	“ “
26	00	“ dry
—	—	
350	20	Broken— Wet ¼ inch on pressure face.

3 inches thick
60 lbs. pressure

No. 19—1 Regr'd Cem. : 3 S.

No. Hrs.	No. ccs. passed	Remarks
16	00	Face damp
8	00	“ “
20	00	“ “
20	00	“ “
9	00	“ “
38	00	“ “
33	00	“ “
17	00	“ “
7	00	“ “
41	00	“ “
30	00	“ “
43	00	“ “
42	00	“ “
26	00	“ “
—	—	
350	00	Broken— Slightly damp.

6 inches thick
60 lbs. pressure

No. 20—1 Regr'd Cem. : 3 S.

No. Hrs.	No. ccs. passed	Remarks
7	3	Edge moist
41	50	“ “
30	00	“ “
43	20	“ “
42	00	“ “
26	00	Face dry
—	—	
189	73	Broken— Wet ½ inch on pressure face. Center damp.

6 inches thick
60 lbs. pressure

No. 21—1 Regr'd Cem. : 3 S.

No. Hrs.	No. ccs. passed	Remarks
16	00	Edge damp
8	00	“ “
17	00	“ “
23	00	“ “
26	00	“ “
—	—	
90	00	Broken— Wet ⅛ inch on pressure face.

6 inches thick
60 lbs. pressure

No. 23—1 Regr'd Cem. : 3 S.

No. Hrs.	No. ccs. passed	Remarks
16	16	Edge Damp
8	2	" "
20	1	" "
20	00	" "
9	00	" "
38	00	" "
33	00	" "
17	00	" "
7	00	" "
41	00	" "
30	00	" "
43	00	" "
42	00	" "
26	00	" "
—	—	
350	19	3 inches thick 60 lbs. pressure Broken— Wet 1 inch on pressure face.

No. 49—1 Regr'd C. : 2 S. : 4 St.

No. Hrs.	No. ccs. passed	Remarks
4	52	Edge damp 1½ sq. in. dry
16	18	edge damp 1 sq. in. dry
8	00	Edge damp 1 sq. in. dry
17	00	Edge damp 2 sq. in. dry
23	00	Edge damp 5 sq. in. dry
26	00	Face dry
22	00	" "
—	—	
116	70	6 inches thick 60 lbs. pressure Broken— Wet 1 inch on pressure face.

No. 50—1 Regr'd C. : 2 S. : 4 St.

No. Hrs.	No. ccs. passed	Remarks
16		
8	6	Edge damp
20	2	" "
20	00	" "
9	00	" "
38	00	" "
33	12	" "
17	18	" "
7	00	" "
41	00	" "
30	00	" "
43	00	" "
42	17	" "
26	15	" "
—	—	
350	70	6 inches thick 60 lbs. pressure Broken— Wet 1 inch on pressure face.

No. 51—1 Regr'd C. : 2 S. : 4 St.

No. Hrs.	No. ccs. passed	Remarks
16	18	Face damp
8	00	" "
20	5	" "
20	00	" "
9	00	" "
38	00	" "
33	3	" "
17	15	" "
7	00	" "
41	00	" "
30	00	" "
43	00	" "
42	00	" "
26	00	" "
—	—	
350	41	6 inches thick 60 lbs. pressure Broken— Damp thruout

No. 52—1 Regr'd C. : 2 S : 4 St.

No. Hrs.	No. ccs. passed	Remarks
7	13	Face dry 3 inches thick 60 lbs. pressure
41	88	" "
30	00	" "
43	00	" "
42	00	" "
25	00	" "
189	101	
		Broken— Wet ¼ inch on pressure. Face damp thruout

No. 54—1 Regr'd C. : 2 S : 4 St.

No. Hrs.	No. ccs. passed	Remarks
16	00	Face dry 3 inches thick 60 lbs. pressure
8	00	" "
20	00	" "
20	00	" "
9	00	" "
38	00	" "
33	00	" "
17	00	" "
7	00	" "
41	00	" "
30	00	" "
43	00	" "
42	00	" "
26	00	Edge Moist
350	00	Broken— Wet ½ inch on pressure face. Slightly damp thruout

No. 55—1 Regr'd C. : 3 S : 5 St.

No. Hrs.	No. ccs. passed	Remarks
16	3	Face damp 6 inches thick 60 lbs. pressure
8	00	" "
20	00	" "
20	00	" "
9	00	" "
38	00	" "
33	00	" "
17	00	" "
7	00	" "
41	00	" "
30	00	" "
43	00	" "
42	00	" "
26	00	
350	3	
		Broken— Moist thruout

No. 56—1 Regr'd C. : 3 S : 5 St.

No. Hrs.	No. ccs. passed	Remarks
4	00	Edge damp 6 inches thick 60 lbs. pressure
16	3	" "
8	00	" "
17	00	" "
23	00	" "
26	00	" "
22	00	" "
116	3	
		Broken— Wet 2 inches on pressure face.

No. 57—1 Regr'd C. : 3 S. : 5 St.

No. Hrs.	No. ccs. passed	Remarks
7	00	Edge damp.
41	00	“ “
30	00	“ “
43	00	“ “
42	00	“ “
26	00	“ “
—	—	
189	00	6 inches thick 60 lbs. pressure
		Broken— Wet 1 inch on pressure face damp thruout

No. 58—1 Regr'd C. : 3 S. : 5 St.

No. Hrs.	No. ccs. passed	Remarks
4	161	Edge damp.
16	186	“ “
8	29	“ “
17	40	“ “
23	5	“ “
26	00	“ “
22	00	“ “
—	—	
116	421	3 inches thick 60 lbs. pressure
		Broken— Wet ½ inch on pressure face.

No. 59—1 Regr'd C. : 3 S. : 5 St.

No. Hrs.	No. ccs. passed	Remarks
16	00	Face all dry
8	00	“ “ “
20	00	“ “ “
20	00	“ “ “
9	00	“ “ “
38	00	“ “ “
33	00	“ “ “
17	00	“ “ “
—	—	
161	00	3 inches thick 60 lbs. pressure
		Broken— Wet ½ inch on pressure face, remainder of specimen dry.

No. 7—1 Common Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
16	00	Face dry
8	00	“ “
20	00	“ “
20	00	“ “
9	00	“ “
38	00	“ “
33	00	“ “
17	00	“ “
7	00	“ “
41	00	“ “
30	00	“ “
43	00	“ “
42	00	“ “
26	00	“ “
—	—	
350	00	3 inches thick 60 lbs. pressure
		Broken— Wet 1½ inch on pressure face.

No. 8—1 Common Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
20	28	Face dry
26	2	“ “
22	00	“ “
—	—	
68	30	
		3 inches thick 60 lbs. pressure
		Broken— Wet 2 inches on pressure face.

No. 10—1 Common Cem. : 2 S.

No. Hrs.	No. cc.s passed	Remarks
16	00	Face damp
8	00	“ “
20	00	“ “
20	00	“ “
9	00	“ “
38	00	“ “
33	00	“ “
17	00	“ “
7	00	“ “
41	00	“ “
30	00	“ “
43	00	“ “
42	00	“ “
29	00	“ Dry
—	—	
350	00	
		6 inches thick 60 lbs. pressure
		Broken— Wet to ½ in. of outer face.

No. 12—1 Common Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	00	Edge damp.
16	00	3 sq. in. dry
8	00	“ “
17	00	“ “
23	00	“ “
26	00	“ “
22	00	Dry.
—	—	
116	00	
		6 inches thick 60 lbs. pressure
		Broken— Saturated to 2 inches of outer face.

No. 26—.9C. : .1 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
16	65	Face dry.
8	29	“ “
20	60	“ “
20	27	“ “
9	3	“ “
38	21	“ “
33	00	“ “
17	00	“ “
7	00	“ “
41	00	“ “
30	00	“ “
43	00	“ “
42	00	“ “
29	00	“ “
—	—	
350	205	
		6 Inches thick 60 lbs. pressure
		Broken— Wet 1 inch on press. face, damp thruout

No. 27—.9 C. : .1 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	5	Moist edge
16	115	" "
8	51	" "
17	96	" "
23	128	" "
26	112	" "
22	55	" "
116	562	6 inches thick 60 lbs. pressure Broken— Wet 1½ inches on pressure face.

No. 28—.9 C. : .1 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	19	1½ sq. in. dry area
16	64	" "
8	8	3½ "
17	9	" "
23	00	" "
26	00	" "
22	00	
116	100	3 inches thick 60 lbs. pressure Broken— Wet 1½ inches on pressure face.

No. 33—.85 C : .15 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	47	Edge moist
16	228	" "
8	77	" "
17	138	" wet
23	152	" "
26	156	" "
22	126	" "
116	924	6 inches thick 60 lbs. pressure Broken— Wet 1 inch on pressure face, interior damp.

No. 34—.85 C : .15 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	9	Edge moist
16	70	Nearly dry
8	23	" "
17	49	" "
23	88	" "
26	8	" "
22	00	" "
116	247	3 inches thick 60 lbs. pressure Broken— Wet 1¾ inches on pressure face.

No. 35—.85 C. : .15 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	00	Face dry
16	00	" "
8	00	" "
17	00	" "
23	00	" "
26	00	" "
22	00	" "
116	00	3 inches thick 60 lbs. pressure
		Broken— Wet $1\frac{3}{4}$ inches on pressure face.

No. 36—,85 C. : .15 Clay : 2 S.

No. Hrs.	No. ccs. passed	Remarks
16	235	Edge damp.
8	49	" "
20	71	" "
20	120	" "
9	140	" wet
38	180	" "
33	170	" "
17	600	Face half wet.
151	1565	3 inches thick 60 lbs. pressure
		Broken— Wet 2 inches on pressure face.

No. 37—.9 C. : .1 Clay : 3 S.

No. Hrs.	No. ccs. passed	Remarks
16	120	Face moist.
8	40	" "
20	73	" wet
20	47	" "
9	12	" "
38	75	" "
33	12	" "
17	27	" "
7	00	" "
41	10	" "
30	2	" "
43	20	" "
42	132	" "
26	5	" "
350	575	6 inches thick 50 lbs. pressure
		Broken— Wet thruout

No. 39 .9 C. : .1 Clay : 3 S.

No. Hrs.	No. ccs. passed	Remarks
4	278	Edge moist.
16	110	" dryer
8	62	" nearly dry
17	79	" " "
23	60	" " "
26	27	" " "
22	00	" " "
116	616	6 inches thick 60 lbs. pressure
		Broken— Saturated to $\frac{1}{2}$ inch of outer face.

No. 40—.9 C. : .1 Clay : 3 S.

No. Hrs.	No. ccs. passed	Remarks
16	325	Face moist
8	760	" "
20		
20	1840	" wet
9	855	" "
38		" "
33		" "
17		" "
161	3780	6 inches thick 60 lbs. pressure Broken— Saturated thruout.

No. 44—.85 C. : .15 Clay : 3 S.

No. Hrs.	No. ccs. passed	Remarks
7	95	Edge wet
41	325	" "
30	155	" "
43	185	" "
42	150	" "
26	85	" "
189	995	6 inches thick 60 lbs. pressure Broken— Wet thruout.

No. 45—.85 C. : .15 Clay : 3 S.

No. Hrs.	No. ccs. passed	Remarks
4	00	Edge moist.
16	00	" "
8	00	" "
17	00	" "
23	00	" "
26	00	" "
22	00	" "
116	00	6 inches thick 60 lbs. pressure Broken— Very wet to 2 inches of outer face.

No. 15—1 Regr'd Cem. : 2 S.

No. Hrs.	No. ccs. passed	Remarks
4	00	Face dry
16	00	" "
8	09	" "
17	00	" "
23	00	" "
26	00	" "
22	00	" "
116	00	60 lbs. pressure 3 inches thick Broken— Wet ¼ inch on pressure face.

TENSILE STRENGTH OF BRIQUETTES.

No.		Age.				Description.
		7 days	28 days	60 days	90 days	
1	Max.	220	355	315	395	3 Sand: 1 Common Cement.
1	Min.	185	280	256	365	
1	Avg.	203	325	290	377	
2	Max.	335	515	505	475	3 Sand: 1 Reground Cement.
2	Min.	260	440	405	375	
2	Avg.	303	465	464	437	
3	Max.	190	345	410	370	3 Sand—.9 Common Cement.
3	Min.	175	280	305	310	
3	Avg.	183	313	369	346	.1 Clay.
4	Max.	230	360	430	450	3 Sand—.85 Common Cement.
4	Min.	190	350	360	400	
4	Avg.	213	353	398	412	.15 Clay.
5	Max.	325	500	560	545	2 Sand: 1 Common Cement.
5	Min.	295	420	430	490	
5	Avg.	305	453	496	514	
6	Max.	430	560	615	635	2 Sand: 1 Reground Cement.
6	Min.	275	425	585	585	
6	Avg.	368	478	606	613	
7	Max.	350	460	525	585	2 Sand: .9 Common Cement.
7	Min.	280	—	480	510	
7	Avg.	315	—	505	543	.1 Clay.
8	Max.	250	345	455	512	2 Sand: .85 Common Cement.
8	Min.	210	300	345	440	
8	Avg.	231	419	398	481	.15 Clay.
9	Max.	530	695	705	677	Neat Regronnd Cement.
9	Min.	410	450	650	655	
9	Avg.	481	589	655	669	
10	Max.	590	820	750	880	Neat Common Cement.
10	Min.	470	690	615	650	
10	Avg.	521	780	680	773	
11	Max.	465	715	805	550	Neat .9 Common Cement.
11	Min.	400	670	695	465	
11	Avg.	428	695	743	495	.1 Clay.
12	Max.	440	695	620	860	Neat .85 Common Cement.
12	Min.	380	610	545	725	
12	Avg.	420	663	580	790	.15 Clay.

ABSORPTION TEST OF CUBES 3 INCHES x 3 INCHES x 5 INCHES.

No.	Initial Wt.Gr.	Final Wt.Gr.	Diff. Wt.	Pr. Ct Abs.	Description.
1	840	877	37	4.4	1:3 Com. Cem. Wet thru.
2	840	867	27	3.2	“ “ “ “ “
3	878	895	17	1.9	1:2 “ “ Damp “
4	893	911	18	2.0	“ “ “ “ “
5	937	958	21	2.2	1:2:4 Regr'd Cem. “ “
6	901	928	27	3.0	“ “ “ “ “
7	917	938	21	2.3	1:3:5 “ “ “ “
8	958	995	37	3.8	“ “ “ “ “
9	1030	1055	25	2.4	1:2 “ “ “ ¾ “
10	995	1005	10	1.0	“ “ “ Center dry.
11	922	930	8	0.8	1:3 “ “ Damp ⅓ dry.
12	945	955	10	1.05	“ “ “ Center dry.
13	914	948	34	3.7	1:2—10% Clay Damp thru.
14	907	918	11	1.2	“ “ “ “ “
15	925	966	41	4.4	1:2—15% “ “ “
16	863	908	45	5.2	“ “ “ “ “
17	899	956	57	6.3	1:2:4—10% “ Moist “
18	868	926	58	6.6	“ “ “ “ “
19	924	986	62	6.7	1:2:4—15% “ “ “
20	876	937	61	6.9	“ “ “ “ “
21	846	915	69	8.1	1:3—10% “ “ “
22	820	883	63	7.7	“ “ “ “ “
23	797	854	57	7.1	1:3—15% “ “ “
24	884	938	54	6.1	
25	868	922	54	6.2	1:2:4 Com. Cem. Damp thru.
26	900	952	52	5.8	“ “ “ “ “
27	872	937	65	7.4	1:3:5 “ “ Moist “
28	920	988	68	7.4	“ “ “ “ “

NOTE:—All specimens stored under water for 90 days and made from same mixtures as test specimens.

RECAPITULATION OF TESTS

Mortars: 1:2: In the 1:2 mixtures of common cement the age of the specimen did not seem to have any effect on the permeability. The 30 day and 60 day specimens were reserved for special tests but the 40 day specimens were broken and found quite fully saturated. The special tests showed up well but when broken were also found badly saturated.

The clay specimens improved slightly with age, but all except No. 35, in the initial and special tests were largely saturated. No. 35 was dry on the outer face during the entire test and passed no water, but was wet half way through when broken.

As a whole the clay specimens were less impermeable than the specimens with common cement alone.

The 1:2 mixtures of reground cement were much better than the clay or common cement mixtures. Age had no effect on the reground pieces as the 30 day specimens, Nos. 15 and 17, gave better results than any others. Neither of these leaked at all during the tests.

Mortars 1:3: The 1:3 common cement mortars were all badly saturated in every case, when broken.

The 10 per cent clay mixtures were badly wet and in most cases completely saturated when broken. Age seemed to have no effect on these mixtures. On an average these specimens were slightly better than the 1:3 common cement specimens.

The 15 per cent clay mixtures as a whole were a failure. No. 45 was the only one that passed no water in the special tests but when broken it was found to be wet to within 2 inches of outer face.

The reground 1:3 mixtures were all good examples of impermeability. Every specimen, when broken showed wetting in but slightly.

Concrete: 1:2:4: The concrete mixtures with common cement either leaked badly or were found to be completely saturated when broken.

The 1:2:4, 15 per cent clay specimens became wet on the outer than those made of common cement. All specimens leaked badly and in almost every case were completely saturated when broken.

The 1:2:4, 15 per cent specimens became wet on the outer face almost as soon as the pressure was applied and remained so during the entire test. All specimens were completely saturated when broken.

The 1:2:4 reground cement specimens showed in every case but little penetration of water. No. 53 leaked quite badly at the beginning of the test but when broken it was found to be dry in the interior and showed that the leak was around the edge next to the form: evidently due to imperfect bond.

Concrete: 1:3:5: The common cement specimens of this mixture all leaked badly and when broken were found completely saturated.

The reground cement specimens of this mixture were all good and compared favorably with the 1:2:4 specimens with reground cement.

REMARKS

From the data secured, it will be noticed that leaking took place in a great many cases, while in some instances the amount of leak was nil. This was readily explained when the specimens were broken. The leak in some cases was next to the form, due to imperfect bond between the cement and iron, while in other cases the leak was through the edge of the material, apparently due to a less density here than in the center; in still other instances the water penetrated through the center as readily as through the edge. Taking into consideration the outward appearances during the test and the appearance of the interior when broken, there was little difficulty in drawing conclusions as to the permeability of the different mixtures.

It has been advocated in the use of clay as a water-proofing material, that it unites chemically with the other materials of the mortar or concrete, but in these tests the clay was washed out when the specimens were placed under pressure, showing that only a mechanical combination had been produced. All clay specimens when broken showed very poor bond; in some cases there was a noticeably honey-combed condition. In all these specimens, it was also noticed that leaking increased with time under pressure; in some cases becoming so great as to necessitate removal of the specimen. These conditions are readily explained if we consider the clay only mechanically combined. The clay in this case being an inert material, prevents perfect contact of the cement with the sand or stone.

CONCLUSIONS

In conclusion, we would say that the mixtures, using common cement alone, were all failures from the standpoint of impermeability, except the 1:2 specimens, Nos. 7, 8, 10, and 12. These last while not leaking, were deeply penetrated by the water, rendering it unsafe to recommend them where impermeability is required.

The clay instead of making the specimen more impervious increased the permeability, as was shown by both the pressure and absorption tests. It is our belief, therefore, that sand should be free from clay to obtain the most impermeable mixture.

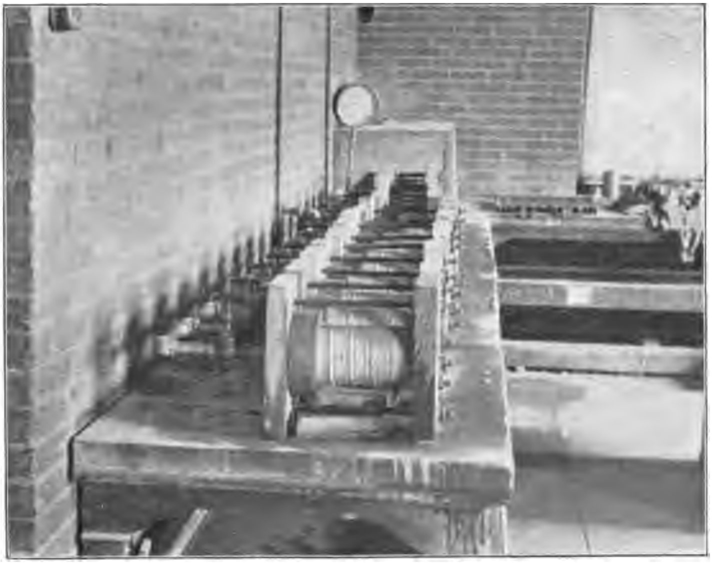
The specimens, in which reground cement was used, showed up well in all three tests. The different mixtures gave fairly

Apparatus for Making Permeability Tests

uniform results and it was not difficult to see their superiority over those with common cement and clay.

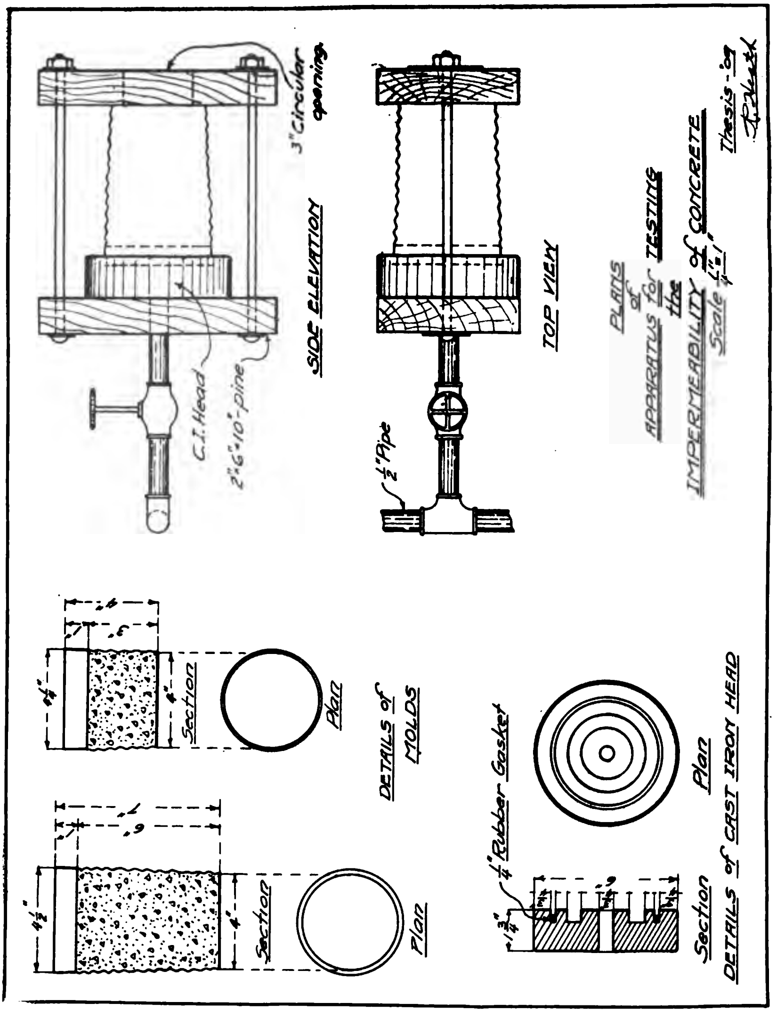
From the results obtained, we would not hesitate in recommending any of the mixtures with reground cement in preference to those with clay or common cement. Not alone does reground cement make a better waterproofing material, but it also increases the strength, as was shown in the tensile tests The fact that reground cement sets more quickly, will not, we believe, be a serious hindrance to its use. When used in mortar or concrete twenty to thirty minutes were required for it to set (initial set), which would be ample time for good workmen to mix and place material.

The greatest hindrance to the use of reground cement is the increased cost of manufacturing, but if the demand were great enough for this material, improved methods of manufacture could probably be devised, by which the cost could be greatly reduced. As to the cost of fine grinding, by the present methods, we would quote from a letter, written by the Edison Portland Cement Co., of Stewartsville, N. J., as follows: "When enough of the trade is ready to pay the increased price,



Apparatus for Making Permeability Tests

we are ready to put the fineness up to 90 per cent or 95 per cent or even better.—we might advise you, however, that grinding from 75 per cent fine to 85 per cent fine costs about as much as grinding from 40 per cent to 75 per cent and that every point you go above 85 per cent increases cost at a greater proportion. To grind 95 per cent would more than double the cost of grinding 75 per cent." This increased cost would be partly off-set by the possibility of using a leaner mixture and yet securing a more impermeable and stronger material.



Details of Permeability Apparatus

TESTS OF PATENT COMPOUNDS FOR WATERPROOFING CONCRETE

BY ROYCE HEATH.

Since the introduction of concrete into modern construction practically all of its properties have been thoroughly studied with the idea of improving the quality of the structure in which it is used. The strength and durability of concrete as a building material is well known. But the important question of how to secure practical impermeability without affecting the other physical properties comes when it is used in the construction of reservoir walls, tanks, dams, aqueducts, etc. through which water must not pass.

A long list of cases might be mentioned where the concrete used in structural work should be waterproofed in order to insure that serious consequences will not follow the lack of protection from moisture. Various methods have been tried and some interesting and valuable data collected along this line. Some of the ways in which concrete has been treated to secure it against the percolation of water are stated in the following general classification,—

A. Application of an exterior coating.

1. Asphalt and tar preparations.
2. Various liquid solutions applied as paints.
3. Mortars rich in cement.

B. Incorporation of some waterproofing substance with the mortar during the process of mixing.

Mixed dry with cement.

1. Clay.
2. Lime.
3. Alum.
4. Patent Compounds.

Mixed with water.

5. Soap.
6. Patent liquid preparations.

Asphalt preparations have been well tried but the fact that asphalt tends to disintegrate when exposed to atmospheric agencies has discouraged its very extensive use. The usual method of application is to spread it on the concrete surface with mops until a thickness of from one-fourth inch to one-half inch has been attained.

Several patent products are sold which are claimed to give an impervious coating. In general these are applied as ordinary paint.

With regard to the use of mortar coatings, the following is quoted from Buell & Hill's Reinforced Concrete:—

"The neatest solution of the problem of securing water tight construction is by all odds the use of an impermeable concrete and setting aside mixtures involving the use of special chemicals this is most nearly secured by means of a wet mixture rich in cement. This construction with the addition of a surfacing of cement mortar is the one that is employed universally in Europe where reinforced concrete tanks and vats are in use in large numbers."

The value of clay has been investigated and is found to be rather doubtful. In certain cases good results have been secured with 10 to 15 per cent of clay mixed dry with the cement. In the majority of cases however the addition of clay was found to be useless.

Mr. R. W. Lesley, Assoc. Mem. Am. Soc. C. E. says of the use of lime,—"The best addition to cement mortars for the purpose of making them impermeable according to the theory of an exterior coating by lime carried through the mass during filtration would be to add to the concrete at the time of mixing a reasonable proportion of hydrate of lime or in other words, the ordinary slacked lime of commerce This slacked lime, as has been shown by experiments of the late Prof. DeSmedt formerly of the laboratory of the District of Columbia does not injure cements or mortars, does not cause expansion and does not decrease their strength though retarding their time of setting."

The statement with regard to the time of setting being retarded was borne out in the tests for this thesis, it being observed that in all cases where powdered compounds were added as a means of water proofing that the time required for setting was materially increased.

Powdered alum has been recommended and several well known writers have mentioned its use in connection with soap solutions in important construction work. Where used it is generally added dry to the cement in quantity varying from one to five per cent. and the soap preparation added to the water used for mixing,—the amount of soap being from one per cent to five per cent of the weight of water.

The introduction of patented compounds is of comparatively recent date although at present there are probably more than twenty varieties on the market. The majority of these are in either powdered or liquid form, to be added to the cement or water before mixing.

Object of the Tests: In this thesis an attempt has been made to determine the value of some of the leading varieties of patent water-proofing compounds by a series of practical tests of specimens in which the substances were used according to the manufacturers' directions.

Specimens for pressure tests were made in two sizes giving concrete 3 inches and 6 inches thick, other dimensions being shown in sketch. Samples were also made with plain concrete in order to get a comparison of results. The highest available pressure was 60 pounds equivalent to a head of about 140 feet.

Besides tests of impermeability against water under different pressures, the same compounds were employed in absorption and tensile tests.

Materials Used in Work: *Cement*: Atlas and Northwestern States brands, which met all requirements of the standard.

Sand: Same as that used in most of college work and by the Experiment Station for testing purposes. Material passing an eighth inch sieve was considered sand, while that retained was classed as gravel.

Gravel: Nothing larger than one inch was used on account of the small size of specimens.

Broken Stone: Iowa limestone passing a ¾ inch mesh and retained on ⅛ inch screen. Thus the stone dust was removed before use.

Waterproofing: The following companies very kindly submitted samples of their product for use in the tests:

A. C. Horn Co., New York City, "Hydratite."

Maumee Chemical Co., Toledo, Ohio, "Maumee."

Miracle Pressed Stone Co., Minneapolis, Minn., "R. O. U."

Sandusky Portland Cement Co., Sandusky, Ohio, "Medusa."

The four compounds above mentioned are prepared in the form of powder to be mixed dry with cement before sand is added.

The National Waterproof Company of Chicago also furnished a sample of "Te-pe-co," a mineral liquid solution applied as paint. The remaining method of waterproofing which was tested was the use of commercial alum and soft soap.

Method of Preparing Specimens for Pressure Tests: The accompanying sketches show the details of the molds. They were made of No. 20 galvanized steel with lap joints and the sides corrugated as shown. The conical shape is for the purpose of preventing the concrete from being forced through by the water pressure and the corrugations were added as a further precaution although they were later found to be unnecessary. All the concrete was prepared in the cement laboratory of Engineering Hall. The temperature of this room is nearly constant at 70 degrees F.

Measurements and Proportion of Waterproofing: For all pressure tests sand, cement, gravel and stone were measured by volume, a metal bowl being used for the purpose. All water-proofing except the soap and alum and liquid preparations were used as 2 per cent of the weight of the cement alone. The soap was used as 1 per cent of the weight of the water while the alum was used as 1 per cent of the weight of the cement. All material was well dried before mixing was commenced.

Mixing: Thi part of the work was done on the stone tables of the laboratory. The molds were placed on a glass plate which had been previously oiled. The cement and waterproofing were first mixed together dry. Care was necessary in this process to secure good results. It was observed that the "Maumee" and "Medusa" compounds mixed readily while the "Hydratite" and "R. O. U." showed a tendency to separate from the cement in small lumps. Sand was next added dry to the cement and waterproofing and enough water to make a rather thin mortar,—finally the stone or gravel was introduced and the whole mixed until quite wet. Enough was usually prepared at a time to fill six molds,—three of each size. Thus two specimens of each mixture were available for the 30, 60 and 90 day tests.

Preparation of Specimens for Absorption Tests: Collapsible wooden forms of ¾ inch pine, having inside dimensions of 2½x2½x4 inches were made for this purpose. These were always oiled before using. Samples for this test were made of a mixture composed of one part cement to three parts of sand by weight. All mixtures were made wet enough to flush water easily when tamped. Later in the work it was decided to select some of the 3 inch specimens for use in absorption tests of concrete. Accordingly fifteen samples were taken to the Masonry Laboratory and air dried for about three months. At the end of this time they had ceased to lose weight.

Storage of Specimens: Those used for pressure tests were kept covered with water during the first month, then wet up once a week afterwards.

Testing Apparatus: Before deciding on the apparatus that was finally used, two other designs were considered but not found to answer the purpose. The first one thought of was to make a hollow casting in the shape of the frustrum of a cone and construct specimens to fit this. The inner surface of the casting was to be machined and the samples of concrete made in molds having the shape of the casting.

The size of these samples would have been as follows:—top diameter, 4½ inches; bottom diameter 4 and thickness 4 inches. The casting was to be provided with a flanged cap having taps for pressure gage, inlet and waste pipes. The idea was abandoned on account of the time required for making the tests since only one sample could have been used at a time.

The second plan was to use a cross or four-way steam fitting provided with taps for gauges and supply pipes. The specimens to be tested were to be made in the form of cylinders 6 inches thick and 13 inches diameter. Bolt holes were provided to correspond to holes in the flanges so that the specimen could be clamped against a gasket and tightened up to prevent the water escaping in any way other than through the concrete. Four test pieces could be used at a time by this method. It was necessary, however, to provide some more expeditious way to do the work and this plan was given up.

The apparatus finally used was designed by the writer. Some modifications were adopted at the suggestion of Professors Marston and Meeker. Complete details are shown in sketch on Page 51. The inlet pipe is at (a) whence the water is led through a one-half galvanized pipe to each of the eighteen stands arranged in a row on one of the stone tables of the laboratory. Each stand is fitted with a gate valve for cutting off the pressure. The remaining parts of the equipment of each stand consist of the two strips 2x6x10 inches (hard pine) used for clamping the blocks, the two bolts ½x13 inches and the cast iron head which is tapped in the center for the inlet pipe. This head is also provided with a circular slot ½ inch deep and ¼ inch wide into which is fitted a heavy rubber ring. The test piece is placed with the top of the mold setting into the slot and then tightened up to prevent leakage at the head.

A pressure gauge placed at one end of the pipe lin and the waste pipe leading from the inlet at (a) to the briquette trays under the tables complete the list of apparatus used. By means of the waste pipe any pressure from 0 to 60 pounds per square inch could be had.

Testing: Three sets of test were made, viz., at 30, 60 and 90 days. In the 30 day test the pressure was placed at 40 pounds during the first twenty-four hours then raised to 60 pounds during the last forty-eight hours of the test. Observations were taken at the end of each period of twenty-four hours.

One problem that caused considerable trouble was the fact that in most of the specimens the pressure would force the water through between the concrete and the inside of the mold and thus prevent any absolute determination of the amount of water passing through the specimen. However, with regard to this difficulty the following points were noted:

1. The leakage from this cause was greatest at the beginning of the test although the pressure may have been as low as twenty pounds.

2. It diminished rapidly and in most casese became less as the pressure increased. The concrete seemed to swell up and fit the mold more closely.

3. In most cases where the concrete could be considered anywhere near water tight the leakage around the edge was practically nil.

4. Nearly every time that the leakage at the edge was rapid the concrete was found to be saturated when broken open at the end of the test.

Results of the briquette test are shown as follows:

TENSION TEST

Table 1

No.	Proportions	Water Proofing	7 day Ave. of 5	28 day Ave. of 4	60 day Ave. of 3	90 day Ave. of 4
1	Neat	"Maumee"	513	490	508	566
2	"	"Hydratite"	494	532	545	502
3	"	"Medusa"	521	640	721	684
4	"	"R. O. U."	489	458	501	540
5	"	None	500	535	602	608
6	1-3	"Maumee"	100	185	230	261
7	"	"Hydratite"	138	172	198	212
8	"	"Medusa"	130	242	306	391
9	"	"R. O. U."	150	182	247	270
10	"	Soap & Alum	145	226	265	316
11	"	None	189	167	250	258

NOTE—Standard sand was used in 1:3 briquettes. All water-proofing used as 2 per cent of the weight of the cement-

From the figures of the above table it will be noticed that the addition of waterproofing does not produce a marked effect upon the tensile strength of a mortar. The only exception is in the case of the first tests of 1:3 briquettes where it appears that the early strength is some whate lessened. However, this effect is too slight to be considered for practical work. The manufacturers claim that no harmful effects will result from the addition of the compound.

The effect of waterproofing upon absorption is shown in the following table:

ABSORPTION TEST OF 1:3 MORTARS— Table 2
(Each result taken as average of two)

No.	Water Proofing	Wt. before immersion in grams	Per cent gain after 1 hour	Per cent gain after 30 days	Remarks. Condition when broken
1	Maumee	790	4.0	7.3	Dry and white inside
2	R. O. U.	775	6.3	8.6	Damp
3	Hydratite	845	2.8	5.5	Dry and white
4	None	805	5.3	7.2	Damp
5	Medusa	787	4.8	7.2	Dry
6	Te-Pe-Co	824	4.4	8.7	Moist
7	Soap & Alum	812	3.8	5.3	Dry

NOTE—Sand used in this test passed No. 8 sieve.

It appears from the above table that the addition of waterproofing does not reduce the percentage of absorption. In fact, in two cases more moisture was taken up by waterproofed blocks than by the plain ones. The conditions under which these specimens were made were the same for each one.

Th absorption of waterproofed concrete is shown in table No. 3 as follows:

ABSORPTION TEST OF CONCRETE Table 3

No.	Proportions				Water proofing	Wt. in grams before immersion	Per cent gain after 1 hour	Per cent gain after 144 hours
	Cement	Sand	Gravel	Stone				
1	1	2		4	Hydra.	1527	2.3	6.9
2	1	2		4	R. O. U.	1531	1.6	5.0
3	1	2		4	None	1575	5.9	7.7
4	1	2		4	Medusa	1501	3.4	7.4
5	1	2		3½	Soap & Alum	1368	3.9	7.9
6	1	2	3½		"	1572	1.8	5.1
7	1	2	3½		Medusa	1433	2.9	6.1
8	1	2	3½		R. O. U.	1517	2.9	5.7
9	1	2	3½		Hydra.	1491	3.5	5.9
10	1	2	3½		Maumee	1505	2.5	5.1
11	1	2		4	Maumee	1488	2.0	5.5

This table shows that the addition of waterproofing retards the absorption of water to some extent at first, but later seems to have a very slight effect.

The data gathered from the pressure tests are arranged in Tables 4, 5, 6 and 7. In some cases the actual amount of water passing was recorded, in others this was found to be useless as nearly all of it was forced through at the edge without passing through the concrete.

GENERAL CONCLUSIONS:

1. A 1:2:4 mixture of broken stone is much more impermeable than a 1:3 mortar.

2. The broken stone mixtures showed up better in all cases than gravel mixtures.

3. Permeability decreases with time, and also as the pressure increases.

4. Concrete seems to shrink in molds when allowed to dry out. This was evidenced a number of times by the ease with which the watter was forced through at the edge of a sample when first put under pressure.

5. The compounds used cannot be considered very satisfactory from the results of the tests. Less than twelve specimens out of about one hundred tested showed that they were practically water tight, and two of these were not water proofed.

It may be that in tests where the pressure would be continuous for several weeks the substances used would be of avail from the fact that the leakage decreases with time.

No sweeping statement can be made with regard to the subject for the extent of this work has been too limited. The writer believes that the problem of waterproofing will be solved by some method other than the addition of chemical compounds.

30 DAY PERMEABILITY TEST

Table 4

Specimen No.	Size	Proportions	Water proofing	After 24 hrs. at 40 lbs.	No. drops in 2 min.	After 48 hrs. 24 to 60 lbs. and 24 to 40 lbs.	After 72 hrs. 24 to 40 lbs. and 48 to 60 lbs.
124	6 inches	1:2:4 stone	Maumee	Moisture just beginning to show thru concrete	2	3 beads show. Rest of surface is dry although discolored	Same as before
130	6 inches	1:2:4 stone	Hydratite	Nearly all dry. No water shows yet	0	About half of surface white and dry. No leakage at all	No leakage at all. Surface still dry
120	6 inches	1:2:4 stone	R. O. U.	Small beads of moisture all over face	18	Leaking very slowly	About same as before
101	6 inches	1:3 mortar	Maumee	Water just appearing all over face	10	Leaking slowly all over surface	Moisture continues to show on surface
136	3 inches	1:2:4 stone	R. O. U.	Wet in three places. Rest of surface damp	10	Leaking all over surface	Considerable less leakage. Surface beginning to dry up
152	3 inches	1:2: 3½ stone	Maumee	No water coming thru concrete. Surface dry	19	Absolutely water tight	Could be classed as water tight. No leakage
155	3 inches	1:2:3½ stone	Hydratite	No water coming through concrete. Surface partly dry	25	Nearly tight. Part of face still dry	No leakage at all. Most of face dry
113	3 inches	1:3 mortar	Maumee	Face all wet. Comes thru concrete	12	Leaking very slowly on surface	Commencing to dry up
127	3 inches	1:2:4 stone	Maumee	Leaks just at edge. Some comes thru concrete	41	Considerable leakage. Water shows all over surface	Continues to leak about the same rate
143	3 inches	1:2:4 stone	None	Face partly dry	2	Nearly tight. Part of face still dry	Very nearly all dry; practically no leakage
104	6 inches	1:3 mortar	R. O. U.	Face is moist	6	Moist all over face	Continues to leak
107	6 inches	1:3 mortar	Hydratite	Moisture showing on face	3	Most of surface wet	Surface stay wet. Constant leakage
146	6 inches	1:3 mortar	None	Small beads on about half of surface	9	Leaking very slowly	Same as before

30 DAY PERMEABILITY TEST. Table 4 continued

Specimen No.	Size	Proportions	Water proofing	After 24 hrs. at 40 lbs.	No. drops in 2 min.	After 48 hrs. 24 to 40 lbs. and 24 to 60 lbs.	After 60 hrs. 24 to 40 lbs. and 48 to 60 lbs.
140	6 inches	1:2:4 stone	None	Water shows on ⅓ surface	8	Continues to leak. Very slowly however	Most of surface dried up. Leakage ceased
133	3 inches	1:2:4 stone	Hydratite	Surface dry. No leakage	0	No leakage. Surface still white	No leakage. Surface dry and white
149	3 inches	1:3 mortar	None	Part in center still dry	2	Practically water tight. Only 1 moist spot	About same as before
161	3 inches	1:2:3½ stone	Medusa	⅓ surface is moist. Rest damp	11	¾ of surface dry. Practically no leakage	Moisture shows again on surface
110	3 inches	1:3 mortar	Hydratite	No water coming thru concrete. Leaks at edge	12	No leakage thru concrete	Beginning to leak thru concrete
165	6 inches	1:3 mortar	Medusa	Not leaking. Only discolored		Water beginning to show thru concrete	Surface remains moist. Wet thru when broken (6)
184	6 inches	1:2:3½ gravel	Medusa	Practically no leakage. Very little at edge		Dryer than before	Much tighter than before. Damp inside when broken (1)
174	6 inches	1:2:3½ stone	Soap and Alum	Leaks constantly all over surface		Leaks rapidly all over surface	Wet and soapy on surface. Soaked inside when broken (4)
187	6 inches	1:2:3½ gravel	R. O. U.	Considerable leakage. Most of surface wet		Continues to leak. Surface wet	Face remains moist. Wet thru when broken (2)
190	6 inches	1:2:3½ gravel	Hydratite	Two wet spots on surface. Rest damp		Water shows constantly	Considerably drier than before. Damp inside when broken
181	6 inches	"	Soap and Alum	Practically water tight		Still dry on surface. No leakage	Surface moist. Wet fracture
168	6 inches	1:2:4 stone	Medusa	No leakage. Part of face white		No moisture shows yet. Dry and white	Practically tight. Most of inside dry when broken

30 DAY PERMEABILITY TEST Table 4 concluded

Specimen No.	Size	Proportions	Water Proofing	After 24 hrs. at 40 lbs.	After 48 hrs. 24 to 40 lbs. and 24 to 60 lbs.	After 72 hrs. 24 to 40 lbs. and 48 to 60 lbs.
193	6 inches	1:2:3½ gravel	Maumee	Very little leakage. Surface damp	Leaks about 1 drop in 2 min.	Dryer than before. Moist inside when broken
207	6 inches	"	None	Surface moist	Getting dryer	Continuing to tighten up Moist inside.
196	3 inches	1:3	Te-Pe-Co	Continues to leak	Rapid leakage	Wet and soapy on surface. Saturated fracture
164	3 inches	1:2:3½ stone	"	Moisture shows considerably	Face all wet	Leaks very fast. Wet fracture
200	3 inches	"	"	Considerable water shows	Leaks rapidly all over	Wet and soapy. Saturated fracture
204	3 inches	1:2:3½ stone	None	Surface wet all the time	Much dryer than before	About 1 drop in 15 min. Fracture only damp
158	3 inches	"	R. O. U.	Nearly water tight. Dry on surface	Water tight. No leakage.	No leakage. Dry fracture
116	3 inches	1:3	R. O. U.	Leaks very slowly. Surface moist	Face continues moist	Still leaks. Wet fracture
171	3 inches	1:2:4 stone	Medusa	Part of face still white and dry	No leakage. Face white.	Practically tight. Moist fracture.
177	3 inches	1:2:3½ stone	Soap and Alum	Leaks rapidly	Continues to leak	Leaks very rapidly. Saturated inside
197	3 inches	1:3	Pe-Pe-Co	Leaks rapidly	No good at all	Soaked all way through

60 DAY PERMEABILITY TEST. TABLE 5.

Specimen No.	Size.	Proportions.	Water-Proofing.	After 24 hrs. at 35 lbs.	Amt. H2O Passing last hr.	After 24 hrs. at 35 and 24 hrs. at 60 lbs.	Amt. H2O Passing last hr.
114	3 inches	1:3	Maumee	Leaks too fast to measure.		Leaks too fast to measure.	
117	3 inches	1:3	R. O. U.	“		“	
128	3 inches	1:2:3 stone	Maumee	“		“	
111	3 inches	1:3	Hydra.	“		“	
156	3 inches	1:2:3½ stone	Hydratite	“		“	
208	3 inches	1:2:3½ gravel	None	7 ccs. Face wet. Concrete leaks.		12 ccs. Face wet and soapy. Comes thru concrete.	
125	6 inches	1:2:4 stone	Maumee	9 ccs. Most leaks thru edge. Face dry and white		12 ccs. Part of face still dry and white.	
102	6 inches	1:3	Maumee	6.5 ccs. Face dry altho discolored.		14 ccs. Leaks at edge. Face dry.	
141	6 inches	1:2:4 stone	None	2 ccs. Face dry. Leaks at edges.		8 ccs. Face moist. Leaks most at edge.	
121	6 inches	“	R. O. U.	6 ccs. Face wet. Concrete leaks.		22 c. c. Face wet and soapy.	
131	6 inches	“	Hydratite	No water passed. Face about halt white.		12 ccs. Surface dry although discolored.	
105	6 inches	1:3	R. O. U.	No water passed. Face dry.		8 ccs. Face a trifle moist.	
147	6 inches	1:3	None	7.5 ccs. Face dry. Leaks at edge.		12 ccs. Face quite dry. Leaks at edge.	
166	6 inches	1:3	Medusa	13.5 ccs. 2 wet spots on face.		11 ccs. Surface wet.	
144	3 inches	1:2:4 stone	None	22 ccs. Surface wet. Leaks considerable.		80 ccs. Surface wet and soapy. Much leakage.	
134	3 inches	“	Hydratite	13 ccs. Surface dry. Water comes through at edges.		20 ccs. Surface damp. Nearly tight.	
150	3 inches	1:3	None	70 ccs. Leaks at edge. Face only trifle moist.		160 ccs. Rapid leakage at edge. Surface moist.	
137	3 inches	1:2:4 stone	R. O. U.	14 ccs. Several leaky spots on surface.		20 ccs. Surface wet and soapy.	
182	6 inches	1:2:3½ gravel	Soap & alum	Part of surface white and dry. Very little leakage at edge.		Two wet spots on face. Saturated fracture when opened.	
191	6 inches	1:2:3½ gravel	Hydratite	½ surface dry. Rest damp. Very slow at edge.		Part of surface dry. Rest damp. No water visible.	
185	6 inches	“	Medusa	⅓ surface white. No moisture shows.		Surface damp. Very little leaks from the edge.	
188	6 inches	“	R. O. U.	Surface damp. Moisture shows in several spots.		Surface most. Saturated fracture.	

60 DAY PERMEABILITY TEST. (Table No. 5 concluded)

Speci-men No.	Size	Propor-tions	Water Proofing	24 hrs. at 35 lbs. pressure	After 24 hrs. at 35 lbs. and 36 hrs. at 60 lbs.
194	6 inches	1:2:3½ gravel	Maumee	Most of surface. dry. Several drops on surface	⅓ surface wet. Rest damp
175	6 inches	1:2;3½ stone	Soap and alum	Face wet. Leaks rapidly at edge	Surface wet and frothy. $CaCO_3$ forced out
206	6 inches	1:2:3½ gravel	None	Surface moist. Leaks rapidly at edge	Surface only moist. Very little leakage at all
169	6 inches	1:2:4 stone	Medusa	Surface dry. Part white. Leaks at edge	½ surface dry and white. Practically no leaks at edge.
201	3 inches	1:2:3½ stone	None	Concrete leaks. Surface wet	Surface wet. Rapid leakage at edge $CaCO_3$ forced out
178	3 inches	“	Soap and alum	Surface wet and frothy. $CaCO_3$ seems to be forced out	Surface wet and soapy. $CaCO_3$ forced out Rapid leakage at edge
172	3 inches	1:2:4 stone	Medusa	About ½ face dry. Rapid leakage at edge	Surface wet and frothy. Rapid leakage at edge
198	3 inches	1:3	None	Face wet and soapy	Surface wet and soapy. Rapid leakage at edge
162	3 inches	1:2:3½ stone	Medusa	½ surface wet. Rest damp. Very slow at edge	Face wet and soapy. Rapid leakage at edge
203	3 inches	“	None	Face all wet. Rapid leakage	Surface wet all over. $CaCO_3$ present
159	3 inches	“	R. O. U.	Surface all wet. Rapid leakage at edge	Surface moist. Wet fracture when opened

90 DAY PERMEABILITY TEST. (Table 6.)

Specimen No.	Size	Proportions	Water Proofing	After 30 hrs. at 30 lbs. pressure	After 30 hrs. at 30 lbs. and 13 hrs. at 60 lbs.
176	6 inches	1:2:3½ stone	Soap and Alum	Surface dry and white. Nery slow at edges.	$CaCO_3$ washed out on surface. Fabe wet. Leaks slowly.
109	6 inches	1:3	Hydratite	Face moist.	Surface wet. Milky water comes through.
106	6 inches	1:3	R. O. U.	Surface damp. Slow leakage at the edge.	Surface moist. Considerable at edge.
170	6 inches	1:2:4 stone	Medusa	Face dry. Part white not discolored.	Surface only damp. Leaks slowly at edge.
103	6 inches	1:3	Maumee	Surface moist.	Face wet. Drips all over.
132	6 inches	1-2-4 stone	Hydratite	Surface all dry.	Surface dry altho discolored Slow drip at edge.
126	6 inches	1:2:4 stone	Maumee	About ½ dry. Rest damp.	⅓ surface still dry. Nearly water tight.
142	6 inches	"	None	Surface dry altho discolored.	Face damp. Water just begins to show.
202	3 inches	"	None	Surface moist.	Face wet and soapy.
138	3 inches	"	R. O. U.	Surface wet.	Wet. Considerable drip.
199	3 inches	1:2:3½ gravel	None	Face all wet. Leaking rapidly.	Wet and frothy. Leads rapidly.
115	3 inches	1:3	Maumee	Surface wet. Considerable leakage.	Leaks too fast to measure.
151	3 inches	"	None	Face only damp. Water not visible.	Moist. Slow drip.
118	3 inches	"	R. O. U.	Face wet and soapy. Leaks fast.	Leaks too fast to measure.
112	3 inches	"	Hydratite	Wet on surface.	Face wet. Leaks too fast to measure.
123	6 inches	1:2:4	R. O. U.	Dry on face. Considerable leak at edge.	$CaCO_3$ forced out in soapy flakes. Considerable leakage

TABLE 7. SPECIAL TEST OF IMPERMEABILITY

of specimens 90 days old which have been under pressure before.

Specimen No.	Size.	Proportions.	Water-Proofing.	After 7 Days at 60 Lbs.
152	3 inches	1:2:3½ stone	Maumee	Face moist. 1 drop per second.
151	3 inches	1:3	None	Surface moist. 2 drops per minute.
178	3 inches	1:2:3½ stone	Soap & Alum	Face covered with $CaCO_3$ which has hardened. No leakage.
149	3 inches	1:3	None	Face most. 4 drops per minute.
110	3 inches	1:3	Hydratite	Face entirely dry. Some $CaCO_3$ hardened. No leakage.
155	3 inches	1:2:3½ stone	Hydratite	Face moist. $CaCO_3$ present; too fast to measure.
133	3 inches	"	Hydratite	Surface moist, covered with red froth. 10 drops per minute.
126	6 inches	"	Maumee	Face dry. Some $CaCO_3$ on it. One drop in five minutes.
147	6 inches	1:3	None	Surface moist. 1 drop per minute.
131	6 inches	1:2:4 stone	Hydratite	Face dry. No leakage.
140	6 inches	"	None	Face moist. $CaCO_3$ present. 7 drops per minute.
191	6 inches	1:2:3½ gravel	Hydratite	Surface moist. 5 drops per minute.
125	6 inches	1:2:4 stone	Maumee	Face dry. No leakage.
185	6 inches	1:2:3½ gravel	Medusa	Face dry. No leakage.

NOTE.—All of above leaked considerably at start. They had been allowed to dry out before the test started.

AN INVESTIGATION OF THE CHANGE IN VOLUME OF CONCRETE DURING THE HARDENING PERIOD

BY C. W. OKEY AND W. H. LECKLITER.

Many well recognized authorities upon masonry and masonry construction assert, that, during the setting and hardening period, concrete changes a slight amount in volume. This was the conclusion drawn from a number of experiments carried on some twenty years ago. The most extensive and reliable of these experiments were performed in the laboratories of the Massachusetts Institute of Technology in 1887, under the supervision of Prof. Swain.

A brief review of these experiments will here be given. The specimens used were five inch cubes. Various mixtures were used; some of neat Portland cement; some of neat Natural cement; and others of a one cement to one sand; one cement to two sand; one cement to three sand; also mixture of both Natural and Portland cement. Brass tacks were set in the cubes and measurements were taken, by a kind of special caliper, to the nearest .001 inch. The cubes were measured at the age of one day, two days, two, four, six, eight, and twelve weeks. One cube of each mixture was placed in water and one in air. Different brands of cement were used and a large number of specimens made. The results agreed very satisfactorily throughout the experiments. The conclusions drawn by Prof. Swain are as follows:

1. Cement mortars in air diminish in linear dimensions to the end of twelve weeks, usually progressively.
2. In water, mortars increase in like manner, but to a less degree.
3. The richest mixture changes the most.
4. Expansion of quick setting cement is the greatest.
5. Contraction for twelve weeks varied from .23 per cent for neat cubes to .08 per cent for the lean mixtures.
6. Expansion varied for a like period from .12 per cent for neat cubes to .04 per cent for lean mixtures.
7. Change in dimensions was the same in all directions for each cube.

There have been, however, no well recognized tests made of concrete with regard to its change of volume during the setting period. The need of such tests has been emphasized lately by building failures, which, it was alleged, were caused by expansion of the concrete floor slabs. In some of these cases it was claimed that rock and gravel concrete slabs did not change appreciably, but that cinder concrete floor slaps would change very noticeably. It was, to determine the rate of change in volume of rock, gravel, and cinder concrete that the investigation herein described was undertaken.

The specimens used in this investigation were approximately 36 inches by 36 inches by 4 inches. They were made with varying proportions as follows:

Slab No. 1. Made January 21, 1909.
One part cement, two sand, and four broken stone.
Slab No. 2. Made January 22, 1909.
One cement, three sand, five broken stone.
Slab No. 3. Made January 22, 1909.
One cement, two sand, four gravel.
Slab No. 4. Made February 12, 1909.
One cement, three sand, five gravel.
Slab No. 5. Made February 12, 1909.
One cement, two sand, and four cinders.
Slab No. 6. Made February 13, 1909.
One cement, three sand, five cinders.

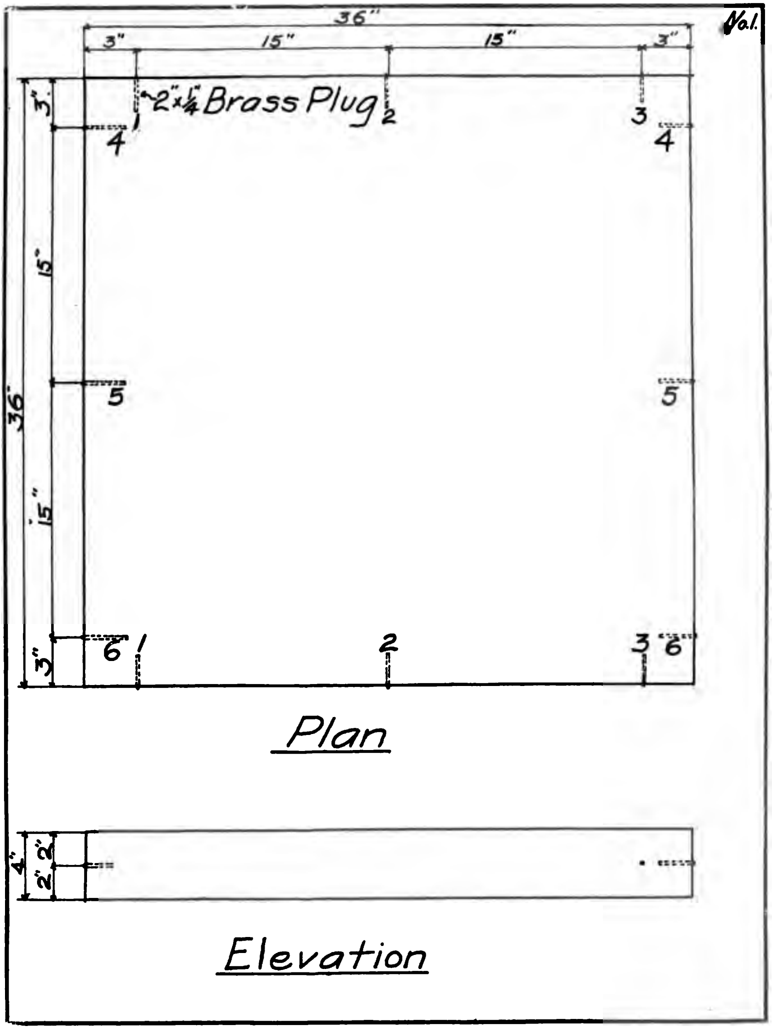
The cement used was North Western States Portland. The cement was tested according to the standard methods, with the following results:

Specific Gravity 3.16.
Fineness. 20.9 per cent retained on 200 sieve.
4.7 per cent retained on 100 sieve.
Normal Consistency, 25 per cent.
Initial Set, two hours and ten minutes.
Final Set, five hours.
Tensile Strength:—

	24 hrs.	7 days.	28 days.
Neat.	110 lbs.	549 lbs.	684 lbs.
1 cement: 3 sand		132 lbs.	281 lbs.

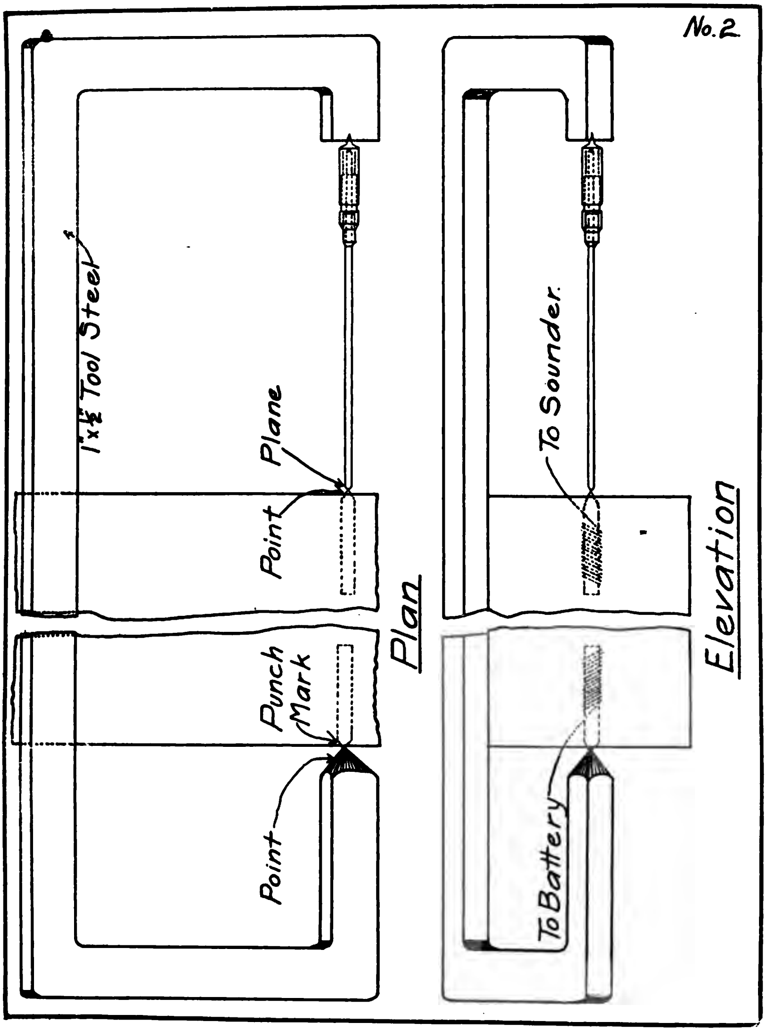
Soundness Test, passed both the Normal and Accelerated tests perfectly.

The sand used was fairly clean and rather sharp. It was passed through a No. 8 sieve to remove all gravel. The gravel used was fairly clean and was of good quality. It was passed over a No. 4 sieve to remove all sand. The crushed rock used was limestone, crushed to about ½ to ¾ inch size. It was



passed over a No. 8 sieve to remove all dust. The cinders used were of average quality and were taken from the college cinder pile. They had been out but a day and had little chance to disintergrate because of exposure. All fine dust and ashes were taken out by passing the cinders over a No. 4 sieve.

The ingredients of the different mixtures of concrete were proportioned by bulk, and were mixed in the usual manner.



The specimens were kept in a room which never varied more than twenty degrees F. in temperature and had an average temperature of 75 degrees F. The specimens were not wet down at any time but were allowed to set with only the water present that was used in mixing the concrete.

The method of measuring change in dimension of the specimens was as follows. In the edge of each slab, three brass plugs were set, as the concrete was being placed in the wooden forms. The location of these plugs is shown in figure No. 1. The special caliper used and method of using it is shown in figure No. 2. As will be noted the large caliper or yoke was made of tool steel and was very rigid. An extensometer, made by the L. C. Starret Co., was used in all measurements. The movement of the micrometer screw was .25 inches, but by the use of rods of known length, the extensometer has a range of several inches. This instrument was calibrated for each rod by the use of the extensometer in the Physics Laboratory. A dry cell and sounder were placed in circuit with the yoke and the extensometer by means of short wires fastened to the brass plugs, set in the slabs. Electrical contact was used in all measurements. Each measurement was checked and average readings taken. By use of a long bar of tool steel and the small extensometer, the spread of the large yoke was often tested and remained constant throughout the experiment.

According to the best authority there is little if any difference in the coefficient of expansion due to temperature changes, of concrete and steel. One good authority gives the difference at .0000005 per degree F. Now with a maximum change of twenty degrees and a length of specimen of three feet, this gives a correction of less than .0004 inches, which is less than the probable error of observation in the regular measurements of the slab. No corrections were made for change in temperature.

The first measurements were taken as soon as they could be taken with safety, that is without dislodging the brass plugs in the slabs. All slabs were measured at the age of one day, and thereafter, as shown in the following tables. In these tables every measurement is given. As will be noted from figure No. 1, there were six separate distances to be measured on each slab. Then for each slab there are six sets of measurements. At the head of each set of measurements the number of the corresponding pair of brass plugs is given. In the first column is the "age in days" of each specimen at the time of taking the corresponding measurement. The second column contains the "number of the rod" used in the caliper for the corresponding measurement.

Now the total length of the caliper, from point to point, for each rod, when the micrometer screw was set at zero, was carefully determined. This is called the "zero length of caliper." The "micrometer reading" of the third column when added to the corresponding "zero length of caliper" of column four will give the terms of column five. The change in length, of the next column, is found by getting the difference between the successive readings in column five, and change per foot is obtained by dividing each term in column No. 6 by three, since three feet was the approximate length of the slab. The last column is obtained by adding the successive changes per foot and thus getting the total change per foot for each age of the slab.

The last sheet is a summary of the other tables. There, is given, the average change per foot for each block along with the corresponding age in days. It is from this table that the average curves are plotted.

The first sheet of curves contains the minimum,. average, and maximum contraction curves for blocks No. 1, 2, 3, and 4. It will be noted that the average contraction of the blocks is pretty uniform from the beginning for about 20 days, and that for the last two, contraction has been nearly uniform throughout. Also, the contraction of the rock concrete is greater than the contraction of the gravel concrete. The greatest average contraction at the age of 91 days for the four blocks was in the second, where a contraction of .0048 inches per foot is shown, or about ½ inch in a hundred feet of concrete. The maximum contraction is also found in that block, .0062 inches per foot. The average contraction for the two gravel blocks, at the same age is about the same, .0035 inches and .0030 inch per foot, and block No. 3, has a minimum contraction of .0015 inch per foot, which is the smallest contraction for the whole number of blocks.

The next curve sheet has the three curves for the first of the cinder concrete slabs. It will be noted that the expansion is rather slow at the beginning but afterwards expands at an increasing rate until an age or 50 days is reached. It then continues to expand at a decreasing rate until the end. A wide variation in amount of expansion is evident, since the total change per foot at an age of 84 days varies from a minimum of .0022 inch per foot to a minimum of .0240 inches per foot.

The last curve sheet gives the three curves for the last block of cinder concrete. In this block the expansion began at once and continued rather uniformly up to the age of 20 days after which it continued to expand at a decreasing rate to the age of 84 days. Here, also, we have a wide variation in the three curves, from expansion at the age of 84 days of .0296 inch for the minimum through an average of .0440 inch to the maximum change per foot of .0532 inch. Here is shown then, at the age of 84 days, an expansion per foot that would give a movement of 4 inches in a concrete floor, a hundred feet long. The difference in the rate of expansion of the two blocks of cinder concrete might be due to one or all of the following conditions: difference in the proportions of the two mixtures of concrete; variability in the character of the cinders; and, difference in the amount of water used in mixing. Block No. 6 was mixed much wetter than No. 5. In this connection it might be well to have a chemical analysis of the cinders, and also a record of the exact amount of water used in the mixing.

Both of the cinder concrete blocks showed numerous checks and cracks, as would be expected, since they had such a great variation in rate of expansion for different parts of each specimen. All of the rock and gravel concrete blocks were perfectly sound. It seems, then, that this unsoundness, these checks and cracks of the cinder concrete blocks, must have been due to the use of cinders as an aggregate. Similarly, it would appear that the expansion of the cinder concrete blocks was due to the cinders. At this point it might be again stated that the cinders used in these blocks were taken from the cinder pile shortly after they were dumped from the furnace, and had not been exposed to the air more than a day.

Although the curves already described, seem to show that the changing in the blocks has almost stopped, the measurements will be continued for some months; or until it is certain that all change has ceased.

In summary then, the investigation has brought out the fact that rock and gravel concrete have a slight contraction while hardening in air, and that cinder concrete has a decided and unmistakeable expansion while hardening in air. Any further investigation of cinder concrete might well include the following additional items: a chemical analysis of all cinders used; a larger number of specimens; duplicate specimens placed in water as well as in air; the use of weathered cinders as well as fresh ones; exact amount of water used in mixing to be recorded; and a number of reinforced concrete specimens.

The essential point of the whole investigation is the fact, that there is a difference between cinder concrete and concrete having rock or gravel as an aggregate. This fact then would seem to warrant an entensive investigation of the whole subject. Such an investigation might well include all the points mentioned above, points which were of necessity omitted from the present investigation.

BLOCK NO. 1

1 Cement : 2 Sand : 4 Broken Stone. Made Jan 21, 1909

Measurements Between Points 1-1

Age in days . . .	Rod No.	Micrometer Reading	Zero Length of Caliper. . . .	Zero Length + Reading . .	Change in Length	Change per ft. .	Summation Change per ft. . . . . .
1	1	0.134"	6.945"	7.079"	0.000"	0.0000"	0.0000"
2	1	0.134"	6.945"	7.079"	0.000"	0.0000"	0.0000"
3	1	0.135"	6.945"	7.080"	—0.001"	—0.0003"	—0.0003"
4	1	0.136"	6.945"	7.081"	—0.001"	—0.0003"	—0.0006"
5	1	0.134"	6.945"	7.079"	+0.002"	+0.0007"	+0.0001"
6	1	0.135"	6.945"	7.080"	—0.001"	—0.0003"	—0.0002"
7	1	0.136"	6.945"	7.081"	—0.001"	—0.0003"	—0.0005"
14	1	0.140"	6.945"	7.085"	—0.004"	—0.0013"	—0.0018"
21	1	0.141"	6.945"	7.086"	—0.001"	—0.0003"	—0.0021"
28	1	0.143"	6.945"	7.088"	—0.002"	—0.0007"	—0.0028"
35	1	0.146"	6.945"	7.091"	—0.003"	—0.0010"	—0.0038"
49	1	0.150"	6.945"	7.095"	—0.004"	—0.0013"	—0.0051"
63	1	0.149"	6.945"	7.094"	+0.001	+0.0003"	—0.0048"
91	1	0.148"	6.945"	7.093"	+0.001	+0.0003"	—0.0045"

Measurements Between Points 2-2

Age in days . . .	Rod No.	Micrometer Reading . . .	Zero Length of Caliper. . . .	Zero Length + Reading . .	Change in Length	Change per ft. .	Summation Change per ft. . . . . .
1	1	0.186"	6.945"	7.131"	0.000"	0.0000"	0.0000"
2	1	0.185"	6.945"	7.130"	+0.001"	+0.0003"	+0.0003"
3	1	0.186"	6.945"	7.131"	—0.001"	—0.0003"	0.0000"
4	4	0.184"	6.945"	7.129"	+0.002"	+0.0007"	+0.0007"
5	1	0.185"	6.945"	7.130"	—0.001"	—0.0003"	+0.0004"
6	1	0.185"	6.945"	7.130"	0.000"	0.0000"	+0.0004"
7	1	0.186"	6.945"	7.131"	—0.001"	—0.0003"	+0.0001"
14	1	0.190"	6.945"	7.135"	—0.004"	—0.0013"	—0.0012"
21	1	0.190"	6.945"	7.135"	0.000"	0.0000"	—0.0012"
28	1	0.192"	6.945"	7.137"	—0.002"	—0.0007"	—0.0019"
35	1	0.193"	6.945"	7.138"	—0.001"	—0.0003"	—0.0022"
49	1	0.193"	6.945"	7.138"	0.000"	0.0000"	—0.0022"
63	1	0.195"	6.945"	7.140"	—0.002"	—0.0007"	—0.0029"
91	1	0.194"	6.945"	7.139"	+0.001"	+0.0003"	—0.0026"

Measurements Between Points 3-3

Age in days . . .	Rod No.	Micrometer Reading	Zero Length of Caliper. . . .	Zero Length + Reading . .	Change in Length	Change per ft. .	Summation Change per ft. . . . . .
1	2	0.124"	7.169"	7.293"	0.000"	0.0000"	0.0000"
2	2	0.125"	7.169"	7.294"	—0.001"	—0.0003"	—0.0003"
3	2	0.125"	7.169"	7.294"	0.000"	0.0000"	—0.0003"
4	2	0.126"	7.169"	7.295"	—0.001"	—0.0003"	—0.0006"
5	2	0.126"	7.169"	7.295"	0.000"	0.0000"	—0.0006"
6	2	0.125"	7.169"	7.294"	+0.001"	+0.0003"	—0.0003"
7	2	0.126"	7.169"	7.295"	—0.001"	—0.0003"	—0.0006"
14	2	0.130"	7.169"	7.299"	—0.005"	—0.0013"	—0.0019"
21	2	0.131"	7.169"	7.300"	—0.001"	—0.0003"	—0.0022"
28	2	0.133"	7.169"	7.302"	—0.002"	—0.0007"	—0.0029"
35	2	0.135"	7.169"	7.304"	—0.002"	—0.0007"	—0.0036"
49	2	0.136"	7.169"	7.305"	—0.001"	—0.0003"	—0.0039"
63	2	0.136"	7.169"	7.305"	0.000"	0.0000"	—0.0039"
91	2	0.137"	7.169"	7.306"	—0.001"	—0.0003"	—0.0042"

BLOCK NO. 1

1 Cement : 2 Sand : 4 Broken Stone. Made Jan 21, 1909

Measurements Between Points 4-4

Age in days...	Rod No.........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.035"	6.945"	6.980"	0.000"	0.0000"	0.0000"
2	1	0.035"	6.945"	6.980"	0.000"	0.0000"	0.0000"
3	1	0.035"	6.945"	6.980"	0.000"	0.0000"	0.0000"
4	1	0.037"	6.945"	6.982"	−0.002"	−0.0007"	−0.0007"
5	1	0.036"	6.945"	6.981"	+0.001"	+0.0003"	−0.0004"
6	1	0.036"	6.945"	6.981"	0.000"	0.0000"	−0.0004"
7	1	0.036"	6.945"	6.981"	0.000"	0.0000"	−0.0004"
14	1	0.037"	6.945"	6.982"	−0.001"	−0.0003"	−0.0007"
21	1	0.039"	6.945"	6.984"	−0.002"	−0.0007"	−0.0014"
28	1	0.040"	6.945"	6.985"	−0.001"	−0.0003"	−0.0017"
35	1	0.044"	6.945"	6.989"	−0.004"	−0.0013"	−0.0030"
49	1	0.045"	6.945"	6.990"	−0.001"	−0.0003"	−0.0033"
63	1	0.046"	6.945"	6.991"	−0.001"	−0.0003"	−0.0036"
91	1	0.047	6.945"	6.992"	−0.001"	−0.0003"	−0.0039"

Measurements Between Points 5-5

Age in days...	Rod No.........	Micrometer Reading	of Caliper.... Zero Length	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	3	0.220"	7.335"	7.555"	0.000"	0.0000"	0.0000"
2	3	0.220"	7.335"	7.555"	0.000"	0.0000"	0.0000"
3	3	0.221"	7.335"	7.556"	−0.001"	−0.0003"	−0.0003"
4	3	0.220"	7.335"	7.555"	+0.001"	+0.0003"	0.0000"
5	3	0.220"	7.335"	7.555"	0.000"	0.0000"	0.0000"
6	3	0.222"	7.335"	7.557"	−0.002"	−0.0007"	−0.0007"
7	3	0.222"	7.335"	7.557"	0.000"	−0.0000"	−0.0007"
14	3	0.229"	7.335"	7.564"	−0.007"	−0.0023"	−0.0030"
21	3	0.233"	7.335"	7.568"	−0.004"	−0.0013"	−0.0043"
28	3	0.237"	7.335"	7.572"	−0.004"	−0.0013"	−0.0056"
35	3	0.236"	7.335"	7.571"	+0.001"	+0.0003"	−0.0053"
49	3	0.236"	7.335"	7.571"	0.000"	0.0000"	−0.0053"
63	3	0.237"	7.335"	7.572"	−0.001"	−0.0003"	−0.0056"
91	3	0.237"	7.335"	7.572"	0.000"	0.0000"	−0.0056"

Measurements Between Points 6-6

Age in days...	Rod No.........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.103"	7.169"	7.272"	0.000"	0.0000"	0.0000"
2	2	0.103"	7.169"	7.272"	0.000"	0.0000"	0.0000"
3	2	0.104"	7.169"	7.273"	−0.001"	−0.0003"	−0.0003"
4	2	0.105"	7.169"	7.274"	−0.001"	−0.0003"	−0.0006"
5	2	0.104"	7.168"	7.273"	+0.001"	+0.0003"	−0.0003"
6	2	0.104	7.169"	7.273"	0.000"	0.0000"	−0.0003"
7	2	0.104"	7.169"	7.273"	0.000"	0.0000"	−0.0003"
14	2	0.106"	7.169"	7.275"	−0.002"	−0.0007"	−0.0010"
21	2	0.108"	7.169"	7.277"	−0.002"	−0.0007"	−0.0017"
28	2	0.110"	7.169"	7.279"	−0.002"	−0.0007"	−0.0024"
35	2	0.112"	7.169"	7.281"	−0.002"	−0.0007"	−0.0031"
49	2	0.112"	7.169"	7.281"	0.000"	0.0000"	−0.0031"
63	2	0.117"	7.169"	7.286"	−0.005"	−0.0017"	−0.0048"
91	2	0.116"	7.169"	7.285"	+0.001"	+0.0003"	−0 0045"

BLOCK NO. 2

1 Cement : 3 Sand : 5 Broken Stone. Made Jan. 22, 1909

Measurements Between Points 1-1

Age in days...	Rod No.......	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.070″	7.169″	7.239″	0.000″	0.0000″	0.0000″
2	2	0.070″	7.169″	7.237″	+0.002″	+0.0007″	+0.0007″
3	2	0.070″	7.169″	7.239″	−0.002″	−0.0007″	0.0000″
4	2	0.068″	7.169″	7.237″	+0.002″	+0.0007″	+0.0007″
5	2	0.069″	7.169″	7.238″	−0.001″	−0.0003″	+0.0004″
6	2	0.070″	7.169″	7.239″	−0.001″	−0.0003″	+0.0001″
7	2	0.071″	7.169″	7.240″	−0.001″	−0.0003″	−0.0002″
14	2	0.074″	7.169″	7.243″	−0.003″	−0.0010″	−0.0012″
21	2	0.074″	7.169″	7.243″	0.000″	0.0000″	−0.0012″
28	2	0.075″	7.169″	7.244″	−0.001″	−0.0003″	−0.0015″
35	2	0.076″	7.169″	7.245″	−0.001″	−0.0003″	−0.0018″
49	2	0.079″	7.169″	7.248″	−0.003″	−0.0010″	−0.0028″
63	2	0.081″	7.169″	7.250″	−0.002″	0.0007″	−0.0035″
91	2	0.081″	7.169″	7.250″	−0.000″	0.0000″	−0.0035″

Measurements Between Points 2-2

Age in days...	Rod No.......	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.060″	7.169″	7.229″	0.000″	0.0000″	0.0000″
2	2	0.059″	7.169″	7.228″	+0.001″	+0.0003″	+0.0003″
3	2	0.058″	7.169″	7.227″	+0.001″	+0.0003″	+0.0006″
4	2	0.057″	7.169″	7.226″	+0.001″	+0.0003″	+0.0009″
5	2	0.057″	7.169″	7.226″	0.000″	0.0000″	+0.0009″
6	2	0.058″	7.169″	7.227″	−0.001″	−0.0003″	+0.0006″
7	2	0.059″	7.169″	7.228″	−0.001″	−0.0003″	+0.0003″
14	2	0.062″	7.169″	7.231″	−0.003″	−0.0010″	−0.0007″
21	2	0.065″	7.169″	7.234″	−0.003″	−0.0010″	−0.0017″
28	2	0.068″	7.169″	7.237″	−0.003″	−0.0010″	−0.0027″
35	2	0.070″	7.169″	7.239″	−0.002″	−0.0007″	−0.0034″
49	2	0.072″	7.165″	7.241″	−0.002″	−0.0007″	−0.0041″
63	2	0.075″	7.165″	7.244″	−0.003″	−0.0010″	−0.0051″
91	2	0.077″	7.169″	7.246″	−0.002″	−0.0007″	−0.0058″

Measurements Between Points 3-3

Age in days...	Rod No.......	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.080″	7.169″	7.249″	0.000″	0.0000″	0.0000″
2	2	0.082″	7.169″	7.251″	−0.002″	−0.0007″	−0.0007″
3	2	0.084″	7.169″	7.253″	−0.002″	−0.0007″	−0.0014″
4	2	0.083″	7.169″	7.252″	+0.001″	+0.0003″	−0.0011″
5	2	0.085″	7.169″	7.254″	−0.002″	−0.0007″	−0.0018″
6	2	0.087″	7.169″	7.256″	−0.002″	−0.0007″	−0.0025″
7	2	0.087″	7.169″	7.256″	0.000″	0.0000″	−0.0025″
14	2	0.088″	7.169″	7.257″	−0.001″	−0.0003″	−0.0028″
21	2	0.089″	7.169″	7.258″	−0.001″	−0.0003″	−0.0031″
28	2	0.091″	7.169″	7.260″	−0.002″	−0.0007″	−0.0038″
35	2	0.093″	7.169″	7.262″	−0.002″	−0.0007″	−0.0045″
49	2	0.095″	7.169″	7.264″	−0.002″	−0.0007″	−0.0052″
63	2	0.096″	7.169″	7.266″	−0.002″	−0.0007″	−0.0059″
91	2	0.096″	7.169″	7.265″	+0.001″	+0.0003″	−0.0056″

BLOCK NO. 2

1 Cement : 3 Sand : 5 Broken Stone. Made Jan. 22, 1909

Measurements Between Points 4-4

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading . .	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.152″	6.945″	7.097″	0.000″	0.0000″	0.0000″
2	1	0.152″	6.945″	7.097″	0.000″	0.0000″	0.0000″
3	1	0.152″	6.945″	7.097″	0.000″	0.0000″	0.0000″
4	1	0.152″	6.945″	7.097″	0.000″	0.0000″	0.0000″
5	1	0.153″	6.945″	7.098″	—0.001″	—0.0003″	—0.0003″
6	1	0.154″	6.945″	7.099″	—0.001″	—0.0003″	—0.0006″
7	1	0.154″	6.945″	7.099″	0.000″	0.0000″	—0.0006″
14	1	0.158″	6.945″	7.103″	—0.004″	—0.0013″	—0.0019″
21	1	0.158″	6.945″	7.103″	0.000″	0.0000″	—0.0019″
28	1	0.160″	6.945″	7.105″	—0.002″	—0.0007″	—0.0026″
35	1	0.162″	6.945″	7.107″	—0.002″	—0.0007″	—0.0033″
49	1	0.164″	6.945″	7.109″	—0.002″	—0.0007″	—0.0040″
63	1	0.167″	6.945″	7.112″	—0.003″	—0.0010″	—0.0050″
91	1	0.170″	6.945″	7.115″	—0.003″	—0.0010″	—0.0060″

Measurements Between Points 5-5

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading . .	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.114″	6.945″	7.059″	0.000″	0.0000″	0.0000″
2	1	0.113″	6.945″	7.058″	+0.001″	+0.0003″	+0.0003″
3	1	0.115″	6.945″	7.060″	—0.002″	—0.0007″	—0.0004″
4	1	0.113″	6.945″	7.058″	+0.002″	+0.0007″	+0.0003″
5	1	0.115″	6.945″	7.060″	—0.002″	—0.0007″	—0.0004″
6	1	0.115″	6.945″	7.060″	0.000″	0.0000″	—0.0004″
7	1	0.116″	6.945″	7.061″	—0.001″	—0.0003″	—0.0007″
14	1	0.119″	6.945″	7.064″	—0.003″	—0.0010″	—0.0017″
21	1	0.120″	6.945″	7.065″	—0.001″	—0.0003″	—0.0020″
28	1	0.122″	6.945″	7.067″	—0.002″	—0.0007″	—0.0027″
35	1	0.123″	6.945″	7.068″	—0.001″	—0.0003″	—0.0030″
49	1	0.124″	6.945″	7.069″	—0.001″	—0.0003″	—0.0033″
63	1	0.123″	6.945″	7.068″	+0.001″	+0.0003″	—0.0030″
91	1	0.123″	6.945″	7.068″	0.000″	0.0000″	—0.0030″

Measurements Between Points 6-6

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading . .	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.126″	6.945″	7.071″	0.000″	0.0000″	0.0000″
2	1	0.125″	6.945″	7.070″	+0.001″	+0.0003″	+0.0003″
3	1	0.127″	6.945″	7.072″	—0.002″	—0.0007″	—0.0004″
4	1	0.126″	6.945″	7.071″	+0.001″	+0.0003″	—0.0001″
5	1	0.127″	6.945″	7.072″	—0.001″	—0.0003″	—0.0004″
6	1	0.126″	6.945″	7.071″	+0.001″	+0.0003″	—0.0001″
7	1	0.127″	6.945″	7.072″	—0.001″	—0.0003″	—0.0004″
14	1	0.131″	6.945″	7.076″	—0.004″	—0.0013″	—0.0017″
21	1	0.132″	6.945″	7.077″	—0.001″	—0.0003″	—0.0020″
28	1	0.132″	6.945″	7.077″	0.000″	0.0000″	—0.0020″
35	1	0.133″	6.945″	7.078″	—0.001″	—0.0003″	—0.0023″
49	1	0.137″	6.945″	7.082″	—0.004″	—0.0013″	—0.0036″
63	1	0.141″	6.945″	7.086″	—0.004″	—0.0013″	—0.0049″
91	1	0.142″	6.945″	7.087	—0.001″	—0.0003″	—0.0052″

BLOCK NO. 3

1 Cement : 2 Sand : 4 Gravel. Made Jan. 22, 1909

Measurements Between Points 1-1

Age in days. . .	Rod No.	Micrometer Reading . . .	Zero Length of Caliper. . .	Zero Length +Reading . .	Change in Length	Change per ft. .	Summation Change per ft. . . . . .
1	2	0.235″	7.169″	7.404″	0.000″	0.0000″	0.0000″
2	2	0.235″	7.169″	7.404″	0.000″	0.0000″	0.0000″
3	2	0.234″	7.169″	7.403″	+0.001″	+0.0003″	+0.0003″
4	2	0.236″	7.169″	7.405″	−0.002″	−0.0007″	−0.0004″
5	2	0.236″	7.169″	7.405″	0.000″	0.0000″	−0.0004″
6	2	0.234″	7.169″	7.403″	+0.002″	+0.0007″	+0.0003″
7	2	0.234″	7.169″	7.403″	0.000″	0.0000″	+0.0003″
14	2	0.234″	7.169″	7.403″	0.000″	0.0000″	+0.0003″
21	2	0.236″	7.169″	7.405″	−0.002″	−0.0007″	−0.0004″
28	2	0.235″	7.169″	7.404″	+0.001″	+0.0003″	−0.0001″
35	2	0.235″	7.169″	7.404″	0.000″	0.0000″	−0.0001″
49	2	0.240″	7.169″	7.409″	−0.005″	−0.0017″	−0.0018″
63	2	0.243″	7.169″	7.412″	−0.003″	−0.0010;	−0.0028″
91	2	0.245″	7.165″	7.414″	−0.002″	−0.0007″	−0.0035″

Measurements Between Points 2-2

Age in days. . .	Rod No.	Micrometer Reading . . .	Zero Length of Caliper. . .	Zero Length +Reading . .	Change in Length	Change per ft. .	Summation Change per ft. . . . . .
1	2	0.161″	7.169″	7.330″	0.000″	0.0000″	0.0000″
2	2	0.161″	7.169″	7.330″	0.000″	0.0000″	0.0000″
3	2	0.164″	7.169″	7.333″	−0.003″	−0.0010″	−0.0010″
4	2	0.164″	7.169″	7.333″	0.000″	0.0000″	−0.0010″
5	2	0.163″	7.169″	7.332″	+0.001″	+0.0003″	−0.0007″
6	2	0.164″	7.169″	7.333″	−0.001″	−0.0003″	−0.0010″
7	2	0.164″	7.169″	7.333″	0.000″	0.0000″	−0.0010″
14	2	0.170″	7.169″	7.339″	−0.006″	−0.0020″	−0.0030″
21	2	0.171″	7.169″	7.340″	−0.001″	−0.0003″	−0.0033″
28	2	0.170″	7.169″	7.339″	+0.001″	+0.0003″	−0.0030″
35	2	0.172″	7.169″	7.341″	−0.002″	−0.0007″	−0.0037″
49	2	0.171″	7.169″	7.340″	+0.001″	+0.0003″	−0.0034″
63	2	0.170″	7.169″	7.339″	+0.001″	+0.0003″	−0.0031″
91	2	0.171″	7.169″	7.340″	−0.001″	−0.0003″	−0.0034″

Measurements Between Points 3-3

Age in days. . .	Rod No.	Micrometer Reading . . .	Zero Length of Caliper. . .	Zero Length +Reading . .	Change in Length	Change per ft. .	Summation Change per ft. . . . . .
1	2	0.159″	7.169″	7.328″	0.000″	0.0000″	0.0000″
2	2	0.161″	7.169″	7.330″	−0.002″	−0.0007″	−0.0007″
3	2	0.163″	7.169″	7.332″	−0.002″	−0.0007″	−0.0014″
4	2	0.160″	7.169″	7.329″	+0.003″	+0.0010″	−0.0004″
5	2	0.163″	7.169″	7.332″	−0.003″	−0.0010″	−0.0014″
6	2	0.163″	7.169″	7.332″	0.000″	0.0000″	−0.0014″
7	2	0.163″	7.169″	7.332″	0.000″	0.0000″	−0.0014″
14	2	0.167″	7.169″	7.336″	−0.004″	−0.0013″	−0.0027″
21	2	0.166″	7.169″	7.335″	+0.001″	+0.0003″	−0.0024″
28	2	0.169″	7.169″	7.338″	−0.003″	−0.0010″	−0.0034″
35	2	0.170″	7.169″	7.339″	−0.001″	−0.0003″	−0.0037″
49	2	0.169″	7.169″	7.338″	+0.001″	+0.0003″	−0.0034″
63	2	0.170″	7.169″	7.339″	−0.001″	−0.0003″	−0.0037″
91	2	0.172″	7.169″	7.341″	−0.002″	−0.0007″	−0.0044″

BLOCK NO. 3

1 Cement : 2 Sand : 4 Gravel. Made Jan. 22, 1909

Measurements Between Points 4-4

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.207"	6.945"	7.152"	0.000"	0.0000"	0.0000"
2	1	0.205"	6.945"	7.150"	+0.002"	+0.0007"	+0.0007"
3	1	0.208"	6.945"	7.153"	−0.003"	−0.0010"	−0.0003"
4	1	0.206"	6.945"	7.151"	+0.002"	+0.0007"	+0.0004"
5	1	0.208"	6.945"	7.153"	−0.002"	−0.0007"	−0.0003"
6	1	0.206"	6.945"	7.151"	+0.002"	+0.0007"	+0.0004"
7	1	0.207"	6.945"	7.152"	−0.001"	−0.0003"	+0.0001"
14	1	0.208"	6.945"	7.153"	−0.001"	−0.0003"	−0.0002"
21	1	0.208"	6.945"	7.153"	0.000"	0.0000"	−0.0002"
28	1	0.211"	6.945"	7.156"	−0.003"	−0.0010"	−0.0012"
35	1	0.214"	6.945"	7.159"	−0.003"	−0.0010"	−0.0022"
49	1	0.212"	6.945"	7.157"	+0.002"	+0.0007"	−0.0015"
63	1	0.212"	6.945"	7.157"	0.000"	0.0000"	−0.0015"
91	1	0.212"	6.945"	7.157"	0.000 ;	0.0000"	−0.0015"

Measurements Between Points 5-5

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.110"	7.169"	7.279"	0.000"	0.0000"	0.0000 ;
2	2	0.109"	7.169"	7.278"	+0.001"	+0.0003"	+0.0003"
3	2	0.110"	7.169"	7.279"	−0.001"	−0.0003"	0.0000"
4	2	0.111"	7.169"	7.280"	−0.001"	−0.0003"	−0.0003"
5	2	0.111"	7.169"	7.280"	0.000"	0.0000"	−0.0003"
6	2	0.110"	7.169"	7.279"	+0.001"	+0.0003"	0.0000"
7	2	0.111"	7.169"	7.280"	−0.001"	−0.0003"	−0.0003"
14	2	0.110"	7.169"	7.279"	+0.001"	+0.0003"	0.0000"
21	2	0.112"	7.169"	7.281"	−0.002"	−0.0007"	−0.0007"
28	2	0.114"	7.169"	7.283"	−0.002"	−0.0007"	−0.0014"
35	2	0.115"	7.169"	7.284"	−0.001"	−0.0003"	−0.0017"
49	2	0.118"	7.169"	7.287"	−0.003"	−0.0010"	−0.0027"
63	2	0.120"	7.169"	7.289"	−0.002"	−0.0007"	−0.0034"
91	2	0.121"	7.169"	7.290"	−0.001"	−0.0003"	−0.0037"

Measurements Between Points 6-6

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper...	Zero Length +Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.152"	6.945"	7.097"	0.000"	0.0000"	0.0000"
2	1	0.155"	6.945"	7.100"	−0.003"	−0.0010"	−0.0010"
3	1	0.156"	6.945"	7.101"	−0.001"	−0.0003"	−0.0013"
4	1	0.155"	6.945"	7.100"	+0.001"	+0.0003"	−0.0010"
5	1	0.156"	6.945"	7.101"	−0.001"	−0.0003"	−0.0013"
6	1	0.158"	6.945"	7.103"	−0.002"	−0.0007"	−0.0020"
7	1	0.157"	6.945"	7.102"	+0.001"	+0.0003"	−0.0017"
14	1	0.156"	6.945"	7.101"	+0.001"	+0.0003"	−0.0014"
21	1	0.157"	6.945"	7.102"	−0.001"	−0.0003"	−0.0017"
28	1	0.159"	6.945"	7.104"	−0.002"	−0.0007"	−0.0024"
35	1	0.159"	6.945"	7.104"	0.000"	0.0000"	−0.0024"
49	1	0.160"	6.945"	7.105"	−0.001"	−0.0003"	−0.0027"
63	1	0.162"	6.945"	7.107"	−0.002"	−0.0007"	−0.0034"
91	1	0.165"	6.945"	7.110"	−0.003"	−0.0010"	−0.0044"

BLOCK NO. 4

1 Cement : 3 Sand : 5 Gravel. Made Feb. 12, 1909

Measurements Between Points 1-1

Rod No.	Age in days	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	1	0.201″	6.945″	7.146″	0.000″	0.0000″	0.0000″
2	1	0.203″	6.945″	7.148″	−0.002″	−0.0007″	−0.0007″
3	1	0.202″	6.945″	7.147″	+0.001″	+0.0003″	−0.0004″
4	1	0.201″	6.945″	7.146″	+0.001″	+0.0003″	−0.0001″
5	1	0.202″	6.945″	7.147″	−0.001″	−0.0003″	−0.0004″
6	1	0.201″	6.945″	7.146″	+0.001″	+0.0003″	−0.0001″
7	1	0.202″	6.945″	7.147″	−0.001″	−0.0003″	−0.0004″
14	1	0.202″	6.945″	7.147″	0.000″	0.0000″	−0.0004″
21	1	0.208″	6.945″	7.153″	−0.006″	−0.0020″	−0.0024″
28	1	0.208″	6.945″	7.153″	0.000″	0.0000″	−0.0024″
35	1	0.210″	6.945″	7.155″	−0.002″	−0.0007″	−0.0031″
49	1	0.209″	6.945″	7.154″	+0.001″	+0.0003″	−0.0028″
63	1	0.211″	6.945″	7.156″	−0.002″	−0.0007″	−0.0035″
91	1	0.212″	6.945″	7.157″	−0.001″	−0.0003″	−0.0038″

Measurements Between Points 2-2

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	2	0.244″	7.169″	7.413″	0.000″	0.0000″	0.0000″
2	2	0.245″	7.169″	7.414″	−0.001″	−0.0003″	−0.0003″
3	2	0.244″	7.169″	7.413″	+0.001″	+0.0003″	0.0000″
4	2	0.244″	7.169″	7.413″	0.000″	0.0000″	0.0000″
5	2	0.243″	7.169″	7.412″	+0.001″	+0.0003″	+0.0003″
6	2	0.243″	7.169″	7.412″	0.000″	0.0000″	+0.0003″
7	2	0.244″	7.169″	7.413″	−0.001″	−0.0003″	0.0000″
14	2	0.245″	7.169″	7.414″	−0.001″	−0.0003″	−0.0003″
21	2	0.246″	7.169″	7.415″	−0.001″	−0.0003″	−0.0006″
28	2	0.248″	7.169″	7.417″	−0.002″	−0.0007″	−0.0013″
35	2	0.250″	7.169″	7.419″	−0.002″	−0.0007″	−0.0020″
49	2	0.251″	7.169″	7.420″	−0.001″	−0.0003″	−0.0023″
63	2	0.252″	7.169″	7.421″	−0.001″	−0.0003″	−0.0026″
91	2	0.253″	7.169″	7.422″	−0.001″	−0.0003″	−0.0029″

Measurements Between Points 3-3

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	2	0.068″	7.169″	7.237″	0.000″	0.0000″	0.0000″
2	2	0.068″	7.169″	7.237″	0.000″	0.0000″	0.0000″
3	2	0.067″	7.169″	7.236″	+0.001″	+0.0003″	+0.0003″
4	2	0.069″	7.169″	7.238″	−0.002″	−0.0007″	−0.0004″
5	2	0.069″	7.169″	7.238″	0.000″	0.0000″	−0.0004″
6	2	0.069″	7.169″	7.238″	0.000″	0.0000″	−0.0004″
7	2	0.070″	7.169″	7.239″	−0.001″	−0.0003″	−0.0007″
14	2	0.072″	7.169″	7.241″	−0.002″	−0.0007″	−0.0014″
21	2	0.073″	7.169″	7.242″	−0.001″	−0.0003″	−0.0017″
28	2	0.075″	7.169″	7.244″	−0.002″	−0.0007″	−0.0024″
35	2	0.074″	7.169″	7.243″	+0.001″	+0.0003″	−0.0021″
49	2	0.075″	7.169″	7.244″	−0.001″	−0.0003″	−0.0024″
63	2	0.076″	7.169″	7.245″	−0.001″	−0.0003″	−0.0027″
91	2	0.077″	7.169″	7.246″	−0.001″	−0.0003″	−0.0030″

BLOCK NO. 4

1 Cement : 3 Sand : 5 Gravel. Made Feb. 12, 1909

Measurements Between Points 4-4

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	5	0.116"	6.744"	6.860"	0.000"	0.0000"	0.0000"
2	5	0.116"	6.744"	6.860"	0.000"	0.0000"	0.0000"
3	5	0.118"	6.744"	6.862"	−0.002"	−0.0007"	−0.0007"
4	5	0.118"	6.744"	6.862"	0.000"	0.0000"	−0.0007"
5	5	0.117"	6.744"	6.861"	+0.001"	+0.0003"	−0.0004"
6	5	0.118"	6.744"	6.862"	−0.001"	−0.0003"	−0.0007"
7	5	0.117"	6.744"	6.861"	+0.001"	+0.0003"	−0.0004"
14	5	0.119"	6.744"	6.863"	−0.002"	−0.0007"	−0.0011"
21	5	0.120"	6.744"	6.864"	−0.001"	−0.0003"	−0.0014"
28	5	0.120"	6.744"	6.864"	0.000"	0.0000"	−0.0014"
35	5	0.122"	6.744"	6.866"	−0.002"	−0.0007"	−0.0021"
49	5	0.122"	6.744"	6.866"	0.000"	0.0000"	−0.0021"
63	5	0.123"	6.744"	6.867"	−0.001"	−0.0003"	−0.0024"
91	5	0.123"	6.744"	6.867"	0.000"	0.0000"	−0.0024"

Measurements Between Points 5-5

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	5	0.160"	6.744"	6.905"	0.000"	0.0000"	0.0000"
2	5	0.158"	6.744"	6.902"	+0.002"	+0.0007"	+0.0007"
3	5	0.159"	6.744"	6.903"	−0.001"	−0.0003"	+0.0004"
4	5	0.158"	6.744"	6.902"	+0.001"	+0.0003"	+0.0007"
5	5	0.158"	6.744"	6.902"	0.000"	0.0000"	+0.0007"
6	5	0.157"	6.744"	6.901"	+0.001"	+0.0003"	+0.0010"
7	5	0.157"	6.744"	6.901"	0.000"	0.0000"	+0.0010"
14	5	0.159"	6.744"	6.903"	−0.002"	−0.0007"	+0.0003"
21	5	0.160"	6.744"	6.904"	−0.001"	−0.0003"	0.0000"
28	5	0.161"	6.744"	6.905"	−0.001"	−0.0003"	−0.0003"
35	5	0.163"	6.744"	6.907"	−0.002"	−0.0007"	−0.0010"
49	5	0.165"	6.744"	6.909"	−0.002"	−0.0007"	−0.0017"
63	5	0.165"	6.744"	6.909"	0.000"	0.0000"	−0.0017"
91	5	0.167"	6.744"	6.911"	−0.002"	−0.0007"	−0.0024"

Measurements Between Points 6-6

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length + Reading ..	Change in Length	Change per ft..	Summation Change per ft......
1	1	0.109"	6.945"	7.054"	0.000"	0.0000"	0.0000"
2	1	0.108"	6.945"	7.053"	+0.001"	+0.0003"	+0.0003"
3	1	0.110"	6.945"	7.055"	−0.002"	−0.0007"	−0.0004"
4	1	0.109"	6.945"	7.054"	+0.001"	+0.0003"	−0.0001"
5	1	0.110"	6.945"	7.055"	−0.001"	−0.0003"	−0.0004"
6	1	0.110"	6.945"	7.055"	0.000"	0.0000"	−0.0004"
7	1	0.109"	6.945"	7.054"	+0.001"	+0.0003"	−0.0001"
14	1	0.111"	6.945"	7.056"	−0.002"	−0.0007"	−0.0008"
21	1	0.113"	6.945"	7.058"	−0.002"	−0.0007"	−0.0015"
28	1	0.115"	6.945"	7.060"	−0.002"	−0.0007"	−0.0022"
35	1	0.115"	6.945"	7.060"	0.000"	0.0000"	−0.0022"
49	1	0.116"	6.945"	7.061"	−0.001"	−0.0003"	−0.0025"
63	1	0.118"	6.945"	7.063"	−0.002"	−0.0007"	−0.0032"
91	1	0.119"	6.945"	7.064"	−0.001"	−0.0003"	−0.0035"

BLOCK NO. 5

1 Cement : 2 Sand : 4 Cinders. Made Feb. 12, 1909

Measurements Between Points 1-1

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	5	0.130"	6.744"	6.874"	0.000"	0.0000"	0.0000"
2	5	0.131"	6.744"	6.875"	−0.001"	−0.0003"	−0.0003"
3	5	0.130"	6.744"	6.874"	+0.001"	+0.0003"	0.0000"
4	5	0.129"	6.744"	6.873"	+0.001"	+0.0003"	+0.0003"
5	5	0.129"	6.744"	6.873"	0.000"	0.0000"	+0.0003"
6	5	0.128"	6.744"	6.872"	+0.001"	+0.0003"	+0.0006"
7	5	0.129"	6.744"	6.873"	−0.001"	−0.0003"	+0.0003"
14	5	0.128"	6.744"	6.872"	+0.001"	+0.0003"	+0.0006"
21	5	0.128"	6.744"	6.872"	0.000"	0.0000"	+0.0006"
28	5	0.122"	6.744"	6.866"	+0.006"	+0.0020"	+0.0026"
35	5	0.121"	6.744"	6.865"	+0.001"	+0.0003"	+0.0029"
42	5	0.118"	6.744"	6.862"	+0.003"	+0.0010"	+0.0039"
49	5	0.116"	6.744"	6.860"	+0.002"	+0.0007"	+0.0046"
56	5	0.115"	6.744"	6.859"	+0.001"	+0.0003"	+0.0049"
63	5	0.112"	6.744"	6.856"	+0.003"	+0.0010"	+0.0059"
70	5	0.110"	6.744"	6.854"	+0.002"	+0.0007"	+0.0066"
84	5	0.111"	6.744"	6.855"	−0.001"	−0.0003"	+0.0063"

Measurements Between Points 2-2

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	1	0.070"	6.945"	7.015"	0.000"	0.0000"	0.0000"
2	1	0.070"	6.945"	7.015"	0.000"	0.0000"	0.0000"
3	1	0.072"	6.945"	7.017"	−0.002"	−0.0007"	−0.0007"
4	1	0.070"	6.945"	7.015"	+0.002"	+0.0007"	0.0000"
5	1	0.072"	6.945"	7.017"	−0.002"	−0.0007"	−0.0007"
6	1	0.070"	6.945"	7.015"	+0.002"	+0.0007"	0.0000"
7	1	0.069"	6.945"	7.014"	+0.001"	+0.0003"	+0.0003"
14	1	0.065"	6.945"	7.019"	+0.004"	+0.0013"	+0.0016"
21	1	0.059"	6.945"	7.004"	+0.006"	+0.0020"	+0.0036"
28	1	0.055"	6.945"	7.000"	+0.004"	+0.0013"	+0.0049"
35	1	0.053"	6.945"	6.998"	+0.002"	+0.0007"	+0.0056"
42	1	0.049"	6.945"	6.994"	+0.004"	+0.0013"	+0.0069"
49	1	0.047"	6.945"	6.992"	+0.002"	+0.0007"	+0.0076"
56.	1	0.045"	6.945"	6.990"	+0.002"	+0.0007"	+0.0083"
63	1	0.044"	6.945"	6.989"	+0.001"	+0.0003"	+0.0086"
70	1	0.043"	6.945"	6.988"	+0.001"	+0.0003"	+0.0089"
84	1	0.043"	6.945"	6.988"	0.000"	0.0000"	+0.0089"

Measurements Between Points 3-3

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	1	0.160"	6.945"	7.105"	0.000"	0.0000"	0.0000"
2	1	0.161"	6.945"	7.106"	−0.001"	−0.0003"	−0.0003"
3	1	0.160"	6.945"	7.105"	+0.001"	+0.0003"	0.0000"
4	1	0.158"	6.945"	7.103"	+0.002"	+0.0007"	+0.0007"
5	1	0.158"	6.945"	7.103"	0.000"	0.0000"	+0.0007"
6	1	0.160"	6.945"	7.105"	−0.002"	−0.0007"	0.0000"
7	1	0.158"	6.945"	7.103"	+0.002"	+0.0007"	+0.0007"
14	2	0.120"	7.169"	7.289"	+0.002"	+0.0007"	+0.0017"
21	1	0.158"	6.945"	7.103"	0.000"	0.0000"	+0.0007"
28	1	0.158"	6.945"	7.103"	0.000"	0.0000"	+0.0007"
35	1	0.155"	6.945"	7.100"	+0.003"	+0.0010"	+0.0017"
42	1	0.155"	6.945"	7.100"	0.000"	0.0000"	+0.0017"
49	1	0.154"	6.945"	7.099"	+0.001"	+0.0003"	+0.0020"
56	1	0.153"	6.945"	7.098"	+0.001"	+0.0003"	+0.0023"
63	1	0.153"	6.945"	7.098"	0.000"	0.0000"	+0.0023"
70	1	0.153"	6.945"	7.098"	0.000"	0.0000"	+0.0023"
84	1	0.152"	6.945"	7.097"	+0.001"	+0.0003"	+0.0026"

BLOCK NO. 5

1 Cement : 2 Sand : 4 Cinders. Made Feb. 12, 1909

Measurements Between Points 4-4

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading ..	Change in Length	Change per ft..	Summation Change per ft.......
1	2	0.125"	7.169"	7.294"	0.000"	0.0000"	0.0000"
2	2	0.124"	7.169"	7.293"	+0.001"	+0.0003"	+0.0003"
3	2	0.124"	7.169"	7.293"	0.000"	0.0000"	+0.0003"
4	2	0.123"	7.169"	7.292"	+0.001"	+0.0003"	+0.0006"
5	2	0.124"	7.169"	7.293"	−0.001"	−0.0003"	+0.0003"
6	2	0.121"	7.169"	7.290"	+0.003"	+0.0010"	+0.0013"
7	2	0.122"	7.169"	7.291"	−0.001"	−0.0003"	+0.0010"
14	2	0.120"	7.169"	7.289"	+0.002"	+0.0007"	+0.0017"
21	2	0.113"	7.169"	7.282"	+0.007"	+0.0023"	+0.0040"
28	2	0.102"	7.169"	7.271"	+0.011"	+0.0037"	+0.0077"
35	2	0.099"	7.169"	7.268"	+0.003"	+0.0010"	+0.0087"
42	2	0.091"	7.169"	7.260"	+0.008"	+0.0027"	+0.0114"
49	2	0.086"	7.169"	7.255"	+ 0.005"	+0.0017"	+0.0131"
56	2	0.084"	7.169"	7.253"	+0.002"	+0.0007"	+0.0138"
63	2	0.083"	7.169"	7.252"	+0.001"	+0.0003"	+0.0141"
70	2	0.081"	7.169"	7.250"	+0.002"	+0.0007"	+0.0148"
84	2	0.080"	7.169"	7.249	+0.001"	+0.0003"	+0.0151"

Measurements Between Points 5-5

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading ..	Change in Length	Change per ft..	Summation Change per ft.......
1	2	0.081"	7.169"	7.250"	0.000"	0.0000"	0.0000"
2	2	0.081"	7.169"	7.250"	0.000"	0.0000"	0.0000"
3	2	0.082"	7.169"	7.251"	−0.001"	−0.0003"	−0.0003"
4	2	0.081"	7.169"	7.250"	+0.001"	+0.0003"	0.0000"
5	2	0.082"	7.169"	7.251"	−0.001"	−0.0003"	−0.0003"
6	2	0.080"	7.169"	7.249"	+0.002"	+0.0007"	+0.0004"
7	2	0.078"	7.169"	7.247"	+0.002"	+0.0007"	+0.0011"
14	2	0.076"	7.169"	7.245"	+0.002"	+0.0007"	+0.0018"
21	2	0.068"	7.169"	7.237"	+0.008"	+0.0027"	+0.0055"
28	2	0.063"	7.169"	7.232"	+0.005"	+0.0017"	+0.0072"
35	2	0.060"	7.169"	7.229"	+0.003"	+0.0010"	+0.0082"
42	2	0.058"	7.169"	7.227"	+0.002"	+0.0007"	+0.0089"
49	2	0.055"	7.169"	7.224"	+0.003"	+0.0010"	+0.0099"
56	2	0.056"	7.169"	7.225"	−0.001"	−0.0003"	+0.0096"
63	2	0.057"	7.169"	7.226"	−0.001"	−0.0003"	+0.0093"
70	2	0.056"	7.169"	7.225"	+0.001"	+0.0003"	+0.0096"
84	2	0.055"	7.169"	7.224"	+0.001"	+0.0003"	+0.0099"

Measurements Between Points 6-6

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading ..	Change in Length	Change per ft..	Summation Change per ft.......
1	2	0.062"	7.169"	7.231"	0.000"	0.0000"	0.0000"
2	2	0.061"	7.169"	7.230"	+0.001"	+0.0003"	+0.0003"
3	2	0.060"	7.169"	7.229"	+0.001"	+.00003"	+0.0006"
4	2	0.061"	7.169"	7.230"	−0.001"	−0.0003"	+0.0003"
5	2	0.058"	7.169"	7.227"	+0.003"	+0.0010"	+0.0013"
6	2	0.056"	7.169"	7.225"	+0.002"	+0.0007"	+0.0020"
7	2	0.055"	7.169"	7.224"	+0.001"	+0.0003"	+0.0023"
14	2	0.037"	7.169"	7.206"	+0.018"	+0.0060"	+0.0083"
21	2	0.022"	7.169"	7.191"	+0.015"	+.00050"	+0.0133"
28	2	0.011"	7.169"	7.180"	+0.011"	+0.0037"	+0.0170"
35	2	0.005"	7.169"	7.174"	+0.006"	+0.0020"	+0.0190"
42	2	0.000"	7.169"	7.169"	+0.005"	+0.0017"	+0.0207"
49	1	0.222"	6.945"	7.167"	+0.002"	+0.0007"	+0.0214"
56	1	0.220"	6.945"	7.165"	+0.002"	+0.0007"	+0.0221"
63	1	0.218"	6.945"	7.163"	+0.002"	+0.0007"	+0.0228"
70	1	0.216"	6.945"	7.161"	+0.002"	+0.0007"	+0.0235"
84	1	0.215"	6.945"	7.160"	+0.001"	+0.0003"	+0.0238"

BLOCK NO. 6

1 Cement : 3 Sand : 5 Cinders. Made Feb. 13, 1909

Measurements Between Points 1-1

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading . .	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.039″	7,169″	7.268″	0.000″	0.0000″	0.0000″
2	2	0.033″	7.169″	7.202″	+0.006″	+0.0020″	+0.0020″
3	2	0.032″	7.169″	7.201″	+0.001″	+0.0003″	+0.0023″
4	2	0.030″	7.169″	7.199″	+0.002″	+0.0007″	+0.0030″
5	2	0.027″	7.169″	7.196″	+0.003″	+0.0010″	+0.0040″
6	2	0.023″	7.169″	7.192″	+0.004″	+0.0013″	+0.0053″
7	2	0.019″	7.169″	7.188″	+0.004″	+0.0013″	+0.0066″
14	2	0.002″	7.169″	7.171″	+0.017″	+0.0057″	+0.0123″
21	1	0.214″	6.945″	7.159″	+0.012″	+0.0040″	+0.0163″
28	1	0.207″	6.945″	7.152″	+0.007″	+0.0023″	+0.0186″
35	1	0.197″	6.945″	7.142″	+0.010″	+0.0033″	+0.0219″
42	1	0.193″	6.945″	7.138″	+0.004″	+0.0013″	+0.0232″
49	1	0.186″	6.945″	7.131″	+0.007″	+0.0023″	+0.0253″
56	1	0.180″	6.945″	7.125″	+0.006″	+0.0020″	+0.0275″
63	1	0.182″	6.945″	7.127″	−0.002″	−0.0007″	+0.0268″
70	1	0.178″	6.945″	7.123″	+0.004″	+0.0013″	+0.0281″
84	1	0.172″	6.945″	7.117″	+0.004″	+0.0013″	+0.0294″

Measurements Between Points 2-2

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading . .	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.235″	7.169″	7.404″	0.000″	0.0000″	0.0000″
2	2	0.234″	7.169″	7.403″	+0.001″	+0.0003″	+0.0003″
3	2	0.226″	7.169″	7.395″	+0.008″	+0.0027″	+0.0030″
4	2	0.223″	7.169″	7.392″	+0.003″	+0.0010″	+0.0040″
5	2	0.219″	7.169″	7.388″	+0.004″	+0.0013″	+0.0053″
6	2	0.210″	7.169″	7.379″	+0.009″	+0.0030″	+0.0083″
7	2	0.203″	7.169″	7.372″	+0.007″	+0.0023″	+0.0106″
14	2	0.163″	7.169″	7.332″	+0.040″	+0.0133″	+0.0239″
21	2	0.124″	7.169″	7.293″	+0.039″	+0.0130″	+0.0369″
28	2	0.105″	7.169″	7.274″	+0.019″	+0.0063″	+0.0432″
35	2	0.095″	7.169″	7.264″	+0.010″	+0.0033″	+0.0465″
42	2	0.089″	7.169″	7.258″	+0.006″	+0.0020″	+0.0485″
49	2	0.083″	7.169″	7.252″	+0.006″	+0.0020″	+0.0505″
56	2	0.080″	7.169″	7.249″	+0.003″	+0.0010″	+0.0515″
63	2	0.080″	7.169″	7.249″	0.000″	0.0000″	−0.0515″
70	2	0.078″	7.169″	7.247″	+0.002″	+0.0007″	+0.0522″
84	2	0.075″	7.169″	7.244″	+0.003″	+0.0010″	+0.0532″

Measurements Between Points 3-3

Age in days...	Rod No........	Micrometer Reading	Zero Length of Caliper....	Zero Length +Reading . .	Change in Length	Change per ft..	Summation Change per ft......
1	2	0.113″	7.169″	7.282″	0.000″	0.0000″	0.0000″
2	2	0.109″	7.169″	7.278″	+0.004″	+0.0013″	+0.0013″
3	2	0.103″	7.169″	7.272″	+0.006″	+0.0020″	+0.0033″
4	2	0.101″	7.169″	7.270″	+0.002″	+0.0007″	+0.0040″
5	2	0.095″	7.169″	7.264″	+0.006″	+0.0020″	+0.0060″
6	2	0.091″	7.169″	7.260″	+0.004″	+0.0013″	+0.0073″
7	2	0.085″	7.169″	7.254″	+0.006″	+0.0020″	+0.0093″
14	2	0.040″	7.169″	7.209″	+0.045″	+0.0150″	+0.0243″
21	2	0.002″	7.169″	7.171″	+0.038″	+0.0127″	+0.0370″
28	1	0.209″	6.945″	7.154″	+0.017″	+0.0057″	+0.0427″
35	1	0.203″	6.945″	7.148″	+0.006″	+0.0020″	+0.0447″
42	1	0.201″	6.945″	7.146″	+0.002″	+0.0007″	+0.0454″
49	1	0.193″	6.945″	7.138″	+0.008″	+0.0027″	+0.0481″
56	1	0.190″	6.945″	7.135″	+0.003″	+0.0010″	+0.0491″
63	1	0.187″	6.945″	7.132″	+0.003″	+0.0010″	+0.0501″
70	1	0.183″	6.945″	7.128″	+0.004″	+0.0013″	+0.0514″
84	1	0.182″	6.945″	7.127″	+0.001″	+0.0003″	+0.0517″

BLOCK NO. 6

1 Cement : 3 Sand : 5 Cinders. Made Feb. 13, 1909

Measurements Between Points 4-4

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	5	0.070″	6.744″	6.814″	0.000″	0.0000″	0.0000″
2	5	0.068″	6.744″	6.812″	+0.002″	+0.0007″	+0.0007″
3	5	0.065″	6.744″	6.809″	+0.003″	+0.0010″	+0.0017″
4	5	0.063″	6.744″	6.807″	+0.002″	+0.0007″	+0.0024″
5	5	0.060″	6.744″	6.804″	+0.003″	+0.0010″	+0.0034″
6	5	0.055″	6.744″	0.797″	+0.007″	+0.0023″	+0.0057″
7	5	0.047″	6.744″	6.791″	+0.006″	+0.0020″	+0.0077″
14	5	0.001″	6.744″	6.745″	+0.046″	+0.0153″	+0.0230″
21	6	0.201″	6.518″	6.719″	+0.026″	+0.0087″	+0.0317″
28	6	0.184″	6.518″	6.702″	+0.017″	+0.0057″	+0.0374″
35	6	0.176″	6.518″	6.694″	+0.008″	+0.0027″	+0.0401″
42	6	0.169″	6.518″	6.687″	+0.007″	+0.0023″	+0.0424″
49	6	0.162″	6.518″	6.680″	+0.007″	+0.0023″	+0.0447″
56	6	0.156″	6.518″	6.674″	+0.006″	+0.0020″	+0.0467″
63	6	0.151″	6.518″	6.669″	+0.005″	+0.0017″	+0.0484″
70	6	0.146″	6.518″	6.664″	+0.005″	+0.0017″	+0.0501″
84	6	0.142″	6.518″	6.660″	+0.004″	+0.0013″	+0.0514″

Measurements Between Points 5-5

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Change in Length	Change per ft.	Summation Change per ft.
1	5	0.194″	6.744″	6.938″	0.000″	0.0000″	0.0000″
2	5	0.190″	6.744″	6.934″	+0.004″	+0.0013″	+0.0013″
3	5	0.188″	6.744″	6.932″	+0.002″	+0.0007″	+0.0020″
4	5	0.185″	6.744″	6.929″	+0.003″	+0.0010″	+0.0030″
5	5	0.180″	6.744″	6.924″	+0.005″	+0.0017″	+0.0047″
6	5	0.177″	6.744″	6.921″	+0.003″	+0.0010″	+0.0057″
7	5	0.173″	6.744″	6.917″	+0.004″	+0.0013″	+0.0070″
14	5	0.141″	6.744″	6.885″	+0.032″	+0.0107″	+0.0177″
21	5	0.117″	6.744″	6.861″	+0.024″	+0.0080″	+0.0257″
28	5	0.105″	6.744″	6.849″	+0.012″	+0.0040″	+0.0297″
35	5	0.098″	6.744″	6.842″	+0.007″	+0.0023″	+0.0320″
42	5	0.094″	6.744″	6.838″	+0.004″	+0.0013″	+0.0333″
49	5	0.088″	6.744″	6.832″	+0.006″	+0.0020″	+0.0353″
56	5	0.082″	6.744″	6.826″	+0.006″	+0.0020″	+0.0373″
63	5	0.076″	6.744″	6.820″	+0.006″	+0.0020″	+0.0393″
70	5	0.072″	6.744″	6.816″	+0.004″	+0.0013″	+0.0406″
84	5	0.072″	6.744″	6.816″	0.000″	0.0000″	+0.0406″

Measurements Between Points 6-6

Age in days	Rod No.	Micrometer Reading	Zero Length of Caliper	Zero Length + Reading	Changes in Length	Change per ft.	Summation Change per ft.
1	1	0.091″	6.945″	7.036″	0.000″	0.0000″	0.0000″
2	1	0.086″	6.945″	7.031″	+0.005″	+0.0017″	+0.0017″
3	1	0.077″	6.945″	7.022″	+0.009″	+0.0030″	+0.0047″
4	1	0.077″	6.945″	7.022″	0.000″	0.0000″	+0.0047″
5	1	0.075″	6.945″	7.020″	+0.002″	+0.0007″	+0.0054″
6	1	0.072″	6.945″	7.017″	+0.003″	+0.0010″	+0.0064″
7	1	0.067″	6.945″	7.012″	+0.005″	+0.0017″	+0.0081″
14	1	0.040″	6.945″	6.985″	+0.027″	+0.0090″	+0.0171″
21	1	0.016″	6.945″	6.961″	+0.024″	+0.0080″	+0.0251″
28	1	0.005″	6.945″	6.950″	+0.011″	+0.0037″	+0.0288″
35	5	0.199″	6.744″	6.943″	+0.007″	+0.0023″	+0.0311″
42	5	0.192″	6.744″	6.936″	+0.007″	+0.0023″	+0.0334″
49	5	0.190″	6.744″	6.934″	+0.002″	+0.0007″	+0.0341″
56	5	0.188″	6.744″	6.932″	+0.002″	+0.0007″	+0.0348″
63	5	0.186″	6.744″	6.930″	+0.002″	+0.0007″	+0.0355″
70	5	0.183″	6.744″	6.927″	+0.003″	+0.0010″	+0.0365″
84	5	0.179″	6.744″	6.923″	+0.004″	+0.0013″	+0.0378″

SUMMARY OF TABLES

Showing Average Change per foot for Each Block

Block No. 1

Age in days...	Summation Change per ft.....
1	0.0000"
2	0.0000"
3	−0.0002"
4	−0.0003"
5	−0.0001"
6	−0.0002"
7	−0.0004"
14	−0.0016"
21	−0.0021"
28	−0.0029"
35	−0.0035"
49	−0.0038"
63	−0.0043"
91	−0.0042"

Block No. 2

Age in days...	Summation Change per ft.....
1	0.0000"
2	+0.0001"
3	−0.0001"
4	+0.0001"
5	−0.0003"
6	−0.0005"
7	−0.0007"
14	−0.0016"
21	−0.0020"
28	−0.0025"
35	−0.0030"
49	−0.0036"
63	−0.0046"
91	−0.0048"

Block No. 3

Age in days...	Summation Change per ft.....
1	0.0000"
2	−0.0001"
3	−0.0006"
4	−0.0004"
5	−0.0007"
6	−0.0006"
7	−0.0006"
14	−0.0012"
21	−0.0014"
28	−0.0019"
35	−0.0023"
49	−0.0026"
63	−0.0030"
91	−0.0035"

Block No. 4

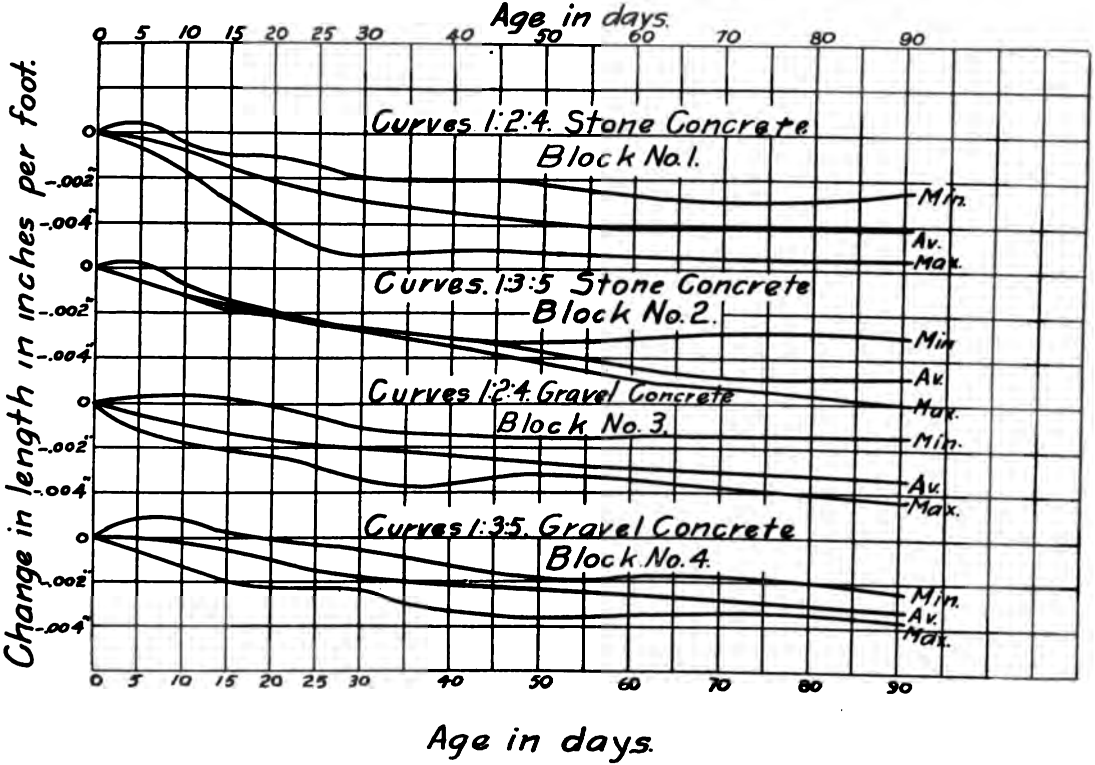
Age in days...	Summation Change per ft.....
1	0.0000"
2	0.0000"
3	−0.0001"
4	−0.0001"
5	−0.0001"
6	−0.0001"
7	−0.0001"
14	−0.0006"
21	−0.0013"
28	−0.0017"
35	−0.0021"
49	−0.0023"
63	−0.0027"
91	−0.0030"

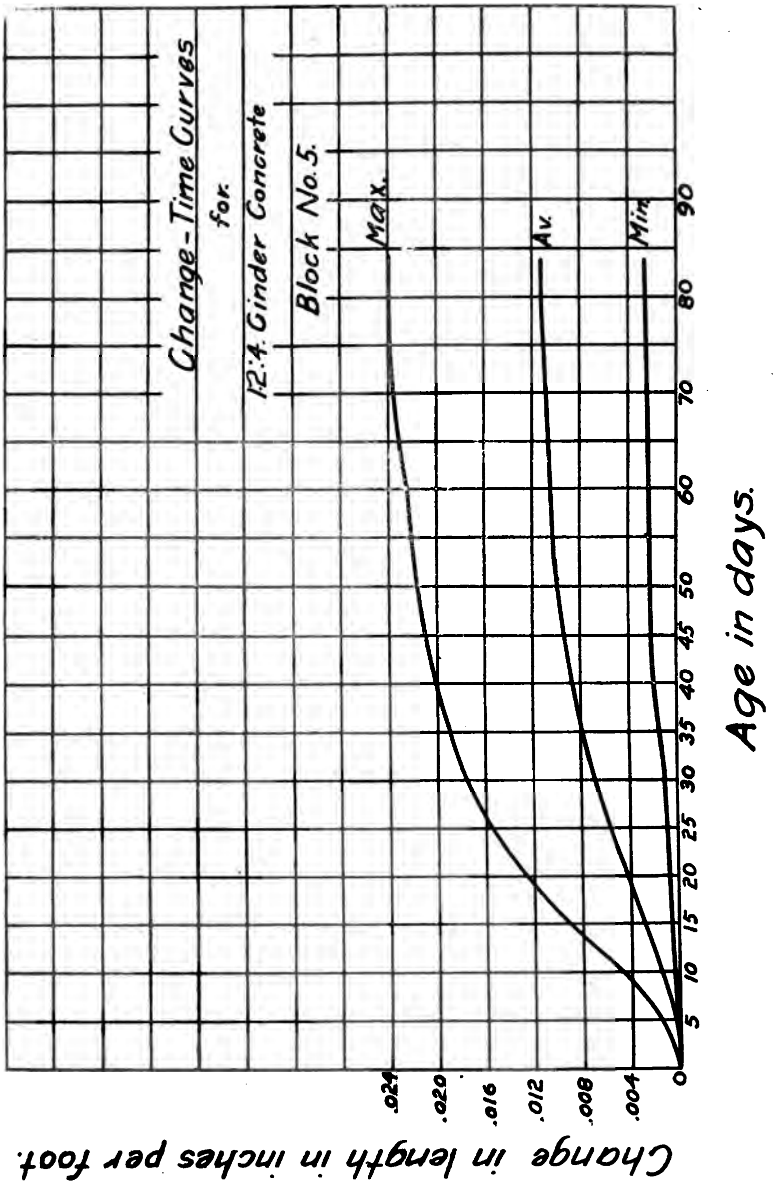
Block No. 5

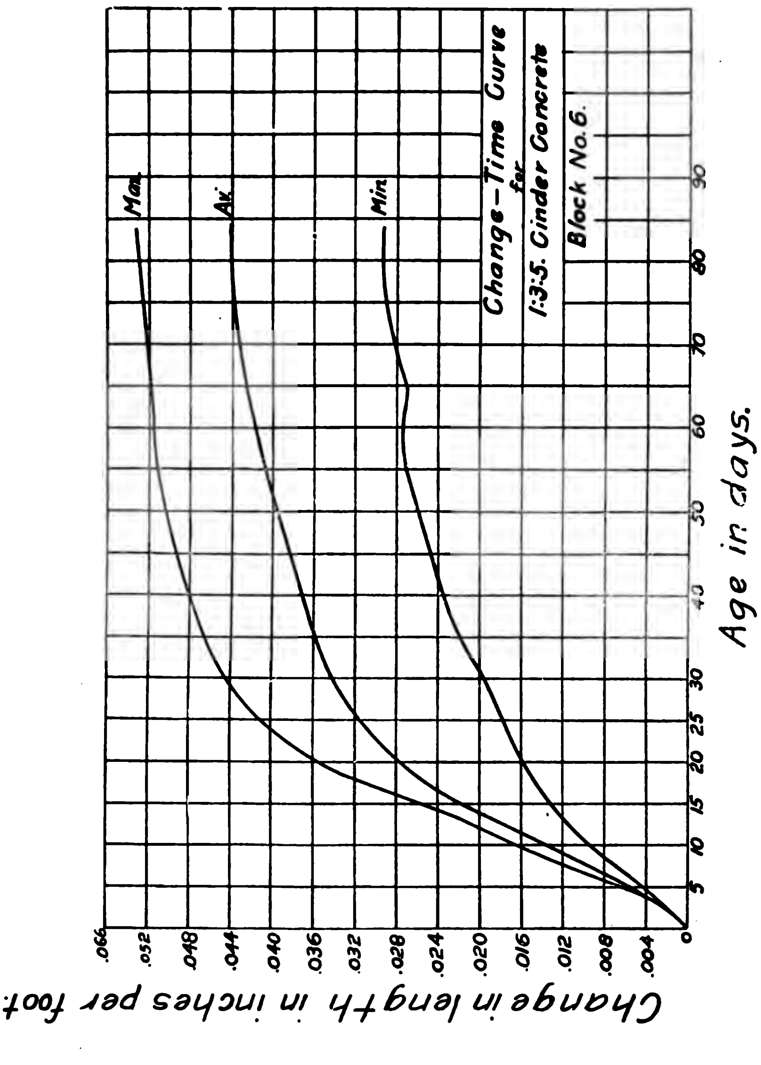
Age in days...	Summation Change per ft.....
1	0.0000"
2	0.0000"
3	0.0000"
4	+0.0003"
5	+0.0003"
6	+0.0009"
7	+0.0009"
14	+0.0024"
21	+0.0046"
28	+0.0067"
35	+0.0077"
42	+0.0089"
49	+0.0098"
56	+0.0102"
63	+0.0105"
70	+0.0109"
84	+0.0111"

Block No. 6

Age in days...	Summation Change per ft.....
1	0.0000"
2	+0.0012"
3	+0.0028"
4	+0.0032"
5	+0.0048"
6	+0.0064"
7	+0.0082"
14	+0.0197"
21	+0.0288"
28	+0.0334"
35	+0.0360"
42	+0.0377"
49	+0.0397"
56	+0.0411"
63	+0.0419"
70	+0.0431"
84	+0.0440"


Age in days.
0
5
10
15
20
25
30
40
50
60
70
80
90
Change in length in inches per foot.
0
-.002"
-.004"
Curves 1:2:4. Stone Concrete
Block No. 1.
Min.
Av.
Max.
Curves. 1:3:5 Stone Concrete
Block No. 2.
Min.
Av.
Max.
Curves 1:2:4. Gravel Concrete
Block No. 3.
Min.
Av.
Max.
Curves 1:3:5. Gravel Concrete
Block No. 4.
Min.
Av.
Max.
Age in days.


Change-Time Curves
for.
1:2:4. Ginder Concrete
Block No. 5.
Max.
Av.
Min.
Change in length in inches per foot.
0
.004
.008
.012
.016
.020
.024
Age in days.
5
10
15
20
25
30
35
40
45
50
60
70
80
90


Change-Time Curve
for
1:3:5. Cinder Concrete
Block No. 6.
Max.
Av.
Min.
Age in days.
Change in length in inches per foot.
0
5
10
15
20
25
30
40
50
60
70
80
90
0
.004
.008
.012
.016
.020
.024
.028
.032
.036
.040
.044
.048
.052
.056

THE COEFFICIENT OF ROUGHNESS IN TILE DRAINS

BY JNO. F. RIGHTMIRE AND MILFORD CHAPPEL.

So far as the authors of this thesis have been able to determine, there have never been any investigations made to determine the value of the coefficient of roughnss, "n", for common drain tile. The value of "n" enters every largely into the design of a tile drainage system. The common practice seems to have been to assume a reasonable value of "n" and go ahead. The assumed value of "n" is based on the valus for "n" for sewer pipe and other surfaces upon which determinations have been made. Since a small change in this factor makes a large change in the value of the constant "c" in the formula, $v = c\sqrt{rs}$ thus materially altering the velocity and the quantity of flow in the tile where $Q = av$ is the quantity of flow, it may easily be seen that a correct value of "n" is of the very greatest importance.

Hundreds of miles of tile are being laid every year at a cost of many thousands of dollars and if these drainage systems are being incorrectly designed, engineers engaged in that kind of work would like to know it, while the men who expect to receive the benefits of the tile drainage should certainly know if their money has been expended wisely.

It is the object of this investigation to secure data at first hand, under actual conditions as they prevail in practice, and to carry out the computations necessary to determine the coefficient of roughness "n" in Kutter's formula and, in a broad way, to try to determine the effect that the size of a tile, the material of which it is made, and the grade and variation in flow, would have on the value of "n".

The work in the field was done after the folllowing general plan. A weir was put in below the outlet of a tile to determine the quantity of flow. The head on the weir was measured on a stake set at the same elevation as the weir notch and about fifteen feet back from the weir. Then up the line of tile from the weir a series of openings were made down to the tile. A small opening was made at the joints. The depth to the water and the depth of the water were measured with a small cedar stick tipped with a sharpened nail. Several sets of readings were taken. The distances between the openings in the tile were measured and levels taken on the openings. The levels were taken to the .001 part of a foot. The distances between the holes to the .1 of a foot and the depth of water and depth to water from the top of the tile were taken to the .01 of an inch. The readings on the weir were taken to the nearest 1 / 16 of an inch.

One example will be discussed in full and the computations carried out to illustrate the method of performing the office work. The eighteen-inch vitrified clay tile at Humboldt, Iowa, will be used as a typical example.

The weir notch was 3.02 feet long. The head on the weir remained constant during the time we took the observations. The head was 3⅝ inches, which, reduced to feet, is .302 of a foot. The quantity of flow was computed from France's formula, which is,

$Q = 3.33\ (b - .2h)h^{3/2}$ where

Q = cubic feet per second

b = length of weir notch

h = head over the weir.

$$Q = 3.33\ (3.02 - .2 \times .302)\ .302^{3/2} = 1.6356 \text{ cu. ft. per sec.}$$

The rod reading on Station 1 was 8.175 feet.

The depth to the water was 12.75 inches; reduced to feet, 1.063 feet.

The sum of these two is the water rod or 9.238 feet. In the same way the water rod for Station 2 is 8.968 feet. The difference in elevation between the points is .27 of a foot. The distance between the stations is 55.4 feet. The hydraulic grade will be .27 / 55.4 or .4873 per cent, or, the average hydraulic grades for this section of the pipe is .2337 per cent. The averages were secured for the other sections in the same manner.

The depth to the water was 12.75 inches; reduced to feet, .613 feet. Next we computed the wetted perimter and the area of flow from the formulas,

$p = \frac{1}{2}\ 3.1416d+d$ into the arc sine $\frac{2d' - d}{d}$

$a = \frac{1}{4}pd + (d' - \frac{1}{2}d)\sqrt{d'(d-d')}$

Where

p = wetted perimeter
a = area of flow
d = diameter of tile
d' = depth of flow.

We assumed that the tile was circular and of the given diameter at each station.

The wetted perimeter at Station 1 is

$p = \frac{1}{2} \times 3.1416 \times 1.5 + 1.5 \text{ arc sin } \frac{2 \times .613 - 1.5}{1.5} = 2.0809$ ft.

and

$a = \frac{1}{4} \times 2.0809 \times 1.5 + (.613 - \frac{1}{2} \times 1.5)\sqrt{.613(1.5 - .613)} = .679359$ ft.

The mean hydraulic radius is the area divided by the wetted perimeter or .6793 ÷ 2.0809 =.3264. The average of these for each station was next secured. Then we figured the velocity of flow in the first section by securing the mean area of flow which was the average of the two end areas of flow:

$Q = av \quad ; \quad v = \frac{Q}{a}$

Where

Q = cu. ft. per sec.
a = area of flow in sq. ft.
v = velocity in ft. per sec.

$v = \frac{1.6356}{.6265} = 2.611$ ft. per second.

$v = C\sqrt{Rs}$

Where

C = a constant
R = hydraulic mean radius
s = sine of the slope.

$$C = \frac{v}{\sqrt{Rs}} = \frac{2.611}{\sqrt{.3115 \times .004219}} = 72.04$$

$$C = \frac{41.66 + \frac{.00281}{s} + \frac{1.811}{n}}{1 + \left(41.66 + \frac{.00281}{s}\right)\frac{n}{\sqrt{R}}}$$

When n = the coefficient of roughness.

$$72\ 04 = \frac{41.66 + \frac{.00281}{.004219} + \frac{1.811}{n}}{1 + \left(41.66 + \frac{.00281}{.004219}\right)\frac{n}{\sqrt{.3115}}}$$

$n = .01568.$

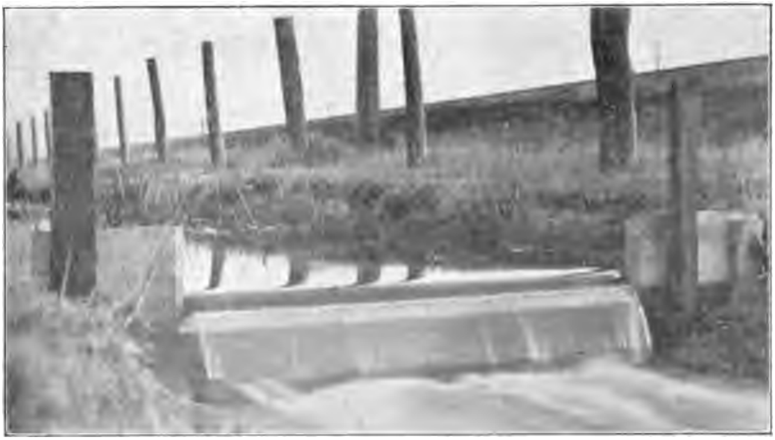
We determined "n" for each section and took the average of all the sections as the "n" for that particular tile.

The following views illustrate the weirs and their location:

View of weir on 14 inch tile at Humboldt, Iowa.

View on 32 inch cement tile at Grand Junction, Iowa.

View on 12 inch clay tile at Humboldt, Iowa.



View of weir on 20 inch cement tile, Grand Junction, Iowa.

View of weir on 18 inch clay tile at Humboldt, Iowa.

In the following tables are compiled the complete data, observations and calculated quantities for the different tiles studied.

32-INCH CEMENT TILE, Grand Junction, Ia.

Sta. Dist.	Average a.	Av. r.	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n.
1	4.4665	.8035						
2	4.3504	.7987	6 0634	1.370	94.48	.0263	.01467	
3	4.5385	.8060		1 364	135.72	.0126	.01047	
4	4.4151	8015		1.353	86.46	.0305	.01592	
5	4.4078	.8013		1.374	89.10	.0297	.01548	.01504

Weir=4 feet. Q=6.0634.

24-INCH CEMENT TILE, Humboldt, Ia.

Sta. Dist.	Average a.	Av. r.	Q cu. ft. sec.	V	D	Av. Hg.	n	Av. n.
1	1.6968	5192	2.077	1.224				
2	1.5627	.4988		1.329	152 9	.0137	.00875	
3	1.6208	5078		1.281	59.6	.0953	.02009	
4	1.6318	.5095		1.272	61 2	.0884	.01967	
5	1.5628	.5007		1.329	71.6	.0653	.01720	

Weir dimension, 3.02 feet, 8 inch notch.

24-INCH CEMENT TILE, Humboldt, Ia. Cont'd

Sta. Dist.	Average a	Av. r	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n
1	1 6469	.5118	2 034	1.235				
2	1.5709	.5038		1 295	100.3	.0313	.01283	
3	1.5709	.5038		1.295	59.7	.0933	.02003	
4	1.5789	.5013		1.288	67.4	.0732	.01807	
5	1.4869	.4862		1.368	87.5	.0467	.01443	.01638

Weir dimension, 3.02 feet. 8 inch notch.

20-INCH CEMENT TILE, Grand Junction, Ia.

Sta. Dist.	Average a	Av. r	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n
1	.9048	3780	1.7055			.0712	.01162	
2	.9883	3949		1.802	108.62	.0870	.01199	
3	.8222	.3588		1.884	104.08	.0812	.01110	
4	.9032	.3603		1.977	114.14	.0855	.01112	
5	.7932	.3516		2.011	113.67			.01146

4 foot Weir.

18-INCH CLAY TILE, Vitrified, Humboldt, Ia.

Sta. Dist.	Average a	Av r	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n
1	.6917	3297	1 6356	2.611	72 04	.4218	.01568	
2	.5613	2933		3 015	83.00	.4591	.01382	.01172
3	.5236	.2817		3.901	145 50	.2938	00860	
4	.3150	2078		5.062	137.78	.6388	.00878	
5	.3312	.2147						

Weir dimension, 3.02 feet wide.

14-INCH CLAY TILE, Vitrified, Humboldt, Ia.

Sta. Dist.	Average a	Av. r	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n
2	.1390	.1370						
3	.0797	.0934	2404	2.198	61.81	01.09	.01414	
4	.1541	1418		2.056	66.53	01.12	.01516	
5	.0631	.0803		2.214	45.70	02.70	.01742	.01525
6	.1633	.1469		2.124	60.79	01.07	.01426	

Weir dimension. 3.02 feet wide.

12-INCH CLAY TILE, Vitrified, Humboldt, Ia.

Sta. Dist.	Average a	Av. r	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n
1	.1219	.1273	.1269	.0816	53.12	.1608	.01655	
2	.1891	.1662		.9410	40 58	.4070	.01976	
3	.0807	.0962		1.418	42.10	1.0837	.01817	.01683
4	.0983	.1112		1.758	65.56	.7915	.01282	
5	.0460	0685						

Weir dimension, 1 foot wide.

10-INCH CLAY TILE, Humboldt, Ia.

Sta. Dist.	Average a	Av. r	Q cu. ft. sec.	V	C	Av. Hg.	n	Av. n
1	.0232	.0464	.01832					
2	.0266	.0531		.7073	28.79	01.213	.0196	.0164
3	.0155	.0355		.8308	35.37	01.246	.0165	
4	.0162	.0366		1.1558	46.04	01.748	.0131	

Weir dimension, 1 foot wide.

This following table is a summary of the work. It gives the size of tile, the material of which it is made, the value of n, the character of the gradient and depth of flow.

Size	Material	n	Character of Grad.	Depth of Flow
2' x 32"	Cement	.01504	Irregular	23½" to 24⅓"
2' x 24"	"	.01638	.01% to .09% Irreg.	½" to 2¾"
2' x 20"	"	.01146	Regular	7¾" to 9½"
2' x 18"	Clay Vit.	.01172	.3% to .6% Regular	4 " to 7½"
2' x 14"	" "	.01525	1% to 2% Irregular	1½" to 3 "
1' x 12"	" "	.01683	.15% to 1.1% Irreg.	1¼" to 2½"
1' x 10"	Clay	.0164	1.2% to 1·7% Fairly Irregular	½" to 1 "

From the table the size of the tile does not seem to influence "n". The length of the tile may have some influence on "n" as the two smallest tile have the largest values. This is probably due to the greater number of joints as the tile over twelve inches in diameter were two feet long.

The depth of flow does not seem to influence "n". This work seems to indicate that irregularities in the grade have the greatest influence on "n". The 20 and 18 inch tile have the lowest values of "n" and they were the two best laid tile in the set. It will require much further work to settle these questions, particularly as to whether the depth of flow does really influence the value of "n". This would require a great many tile of the same diameter laid at different grades flowing at different depths to give sufficient data upon which the base conclusions.

We wish to acknowledge our gratitude to Mr. Floyd Goodrich, County Engineer of Humboldt County, Iowa, and Mr. Thompson of Grand Junction, Iowa, for favors extended when gathering these data.

BULLETINS OF THE ENGINEERING EXPERIMENT STATION

No. 1. The Iowa State College Sewage Disposal Plant Investigations.
No. 2. Bacteriological Investigations of the Iowa State College Sewage.
No. 3. Data of Iowa Sewage and Sewage Disposal.
No. 4. Bacteriological Investigations of the Iowa State College Sewage Disposal Plant.
No. 5. The Chemical Composition of the Sewage of the Iowa State College Sewage Disposal Plant.
No. 6. Tests of Iowa Common Brick.
No. 7. Sewage Disposal in Iowa.
No. 8. Tests of Dry Press Brick Used in Iowa.
No. 9. Notes on Steam Generation with Iowa Coal.
No. 10. Dredging by the Hydraulic Method.
No. 11. An Investigation of Some Iowa Sewage Disposal Systems.
Vol. II. No. 6. The Good Roads Problem in Iowa.
Vol. III. No. 1. Tests of Cement.
Vol. III. No. 2. State Railroad Taxation.
Vol. III. No. 3. Steam Generation with Iowa Coals.
Vol. III. No. 4. Incandescent Lamp Testing.
Vol. III. No. 5. Steam Pipe Covering Tests.
Vol. III. No. 6. The Assessment of Drainage Districts.
Vol. IV. No. 1. Tests of Iowa Limes.
Vol. IV. No. 2. Holding Power of Nails in Single Shear.
Vol. IV. No. 3. Miracle Contest Papers for 1908.
Vol. ~~IV.~~ 5 No. ~~4.~~ 1 Tests and Other Investigations of Cement Drain Tile and Sewer Pipe (In preparation.)
Vol. IV. No. ~~5.~~ 4 Miracle Prize Papers for 1909.
~~Vol. IV. No. 6. Results of Drainage Engineering Investigations for 1908 and 1909.~~
Vol. IV. No. 1. Sanitary Examination of Water Supplies.

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BULLETIN

Vol. IV No. V

THE SANITARY EXAMINATION
OF
WELL WATERS

JANUARY, 1910

INTRODUCTORY

Since the establishment of the Engineering Experiment Station many samples of water have come to the laboratory for examination and many water supplies have been investigated. In this bulletin are compiled some of the data relating to waters from various sources and from different sections of the state. An examination of these results reveals some things of interest, even surprise, to one who is not familiar with the chemical characteristics of a sanitary water or who has not paid particular attention to the conditions surrounding many of our domestic water supplies. That these conditions can often be easily altered so as to render potable a badly polluted and hygienically menacing water supply need not be said. It is believed that the publication of these analyses, with their interpretations from the standpoint of the sanitary value of the water, and the widest possible distribution, are essential steps in the education of the people of the state to an appreciation of the importance of zealously safeguarding the sources of that food element which is most essential to human life; and, be it said, that none other than our waters are more susceptible to contamination, and they are more often than any other the medium for the transmission of disease.

Ira A. Williams, Editor.

THE SANITARY EXAMINATION OF WELL WATERS

BY CHAS. E. ELLIS.

The dangers which exist in the contamination of our domestic water supplies may be very generally known, but they are certainly not regarded as seriously as their importance warrants, nor can attention too often be called to them. Pure and wholesome water in the strict sense is in nature very rare indeed. This is true because of the various conditions met by the moisture of the earth before it becomes available for man's use. Water from rivers which drain large areas of land carries more or less of materials dissolved from the soil and vegetable debris over and through which it has passed. Wells, dug, driven or drilled, to a greater or less depth into the earth tap supplies which have come in contact with soluble minerals of various kinds. Spring waters, though sparkling enough, are yet subject to contamination. The very rains themselves wash and carry down the germ laden dust of the atmosphere. The purest water may be obtained artificially by evaporation and distillation in the laboratory, but such a process is ordinarily impracticable for the home or the city supply.

The usual impurities are in themselves not considered harmful in the quantities in which they exist in the average water. They may, however, be indicative of very serious contamination. Water, being a very powerful solvent, has a tendency to carry with it in solution portions of whatever soluble salts it comes in contact with. Moving water likewise carries particles of matter in suspension. The constant decay of organic matter all about us furnishes most favorable conditions for bacterial growth and affords a most abundant opportunity for the contamination of wells, lakes and streams.

The results of a sanitary water analysis as represented by figures doubtless mean very little to the average layman or even to the City Engineer whose sources of information do not include the most recent books and other publications devoted to such technical subjects. It is the object of this paper to present in as comprehensive terms as may be an explanation of the significance of the determinations made in an analysis of water for sanitary purposes, and to show how the results may be intelligently interpreted. These interpretations are based on the opinions of men of established reputations and of high scientific standing along the lines of water investigations.

It may be well to state here that no "Hard-and fast-Rules" can be applied nor any "set standards" adopted that will be applicable for all cases. But with a knowledge of the surrounding circumstances and an environmental survey of the source of the water in question, an analysis may be of very great value in checking or preventing the spread of epidemics.

The constituents determined in a sanitary water analysis are ordinarily:

Free ammonia	Reported in parts per million.
Albuminoid ammonia	Reported in parts per million.
Chlorine	Reported in parts per million.
Nitrogen as nitrites	Reported in parts per million.
Nitrogen as nitrates	Reported in parts per million.
Oxygen consumed	Reported in parts per million.
Solids on evaporation	Reported in parts per million.
Solids on ignition	Reported in parts per million.
Loss on ignition	Reported in parts per million.

Ammonium compounds are usually results of the putrefactive fermentation of nitrogenous organic matter, or they may be the result of the reduction of nitrites and nitrates in the presence of organic matter. In either case,, contamination is indicated, except in deep wells or where contact with ammonia can be accounted for, as in manufacturing operations where ammonia is used. The presence of much free ammonia is usually considered as indicative of recent contamination, and especially of animal origin.

'Albuminoid ammonia' indicates nitrogenous organic matter not sufficiently decomposed to form ammonium compounds. If high, with chlorine content also high, it is of animal origin and the water should be condemned. It is highly objectionable even if of vegetable origin.

'Chlorine' is a constant and characteristic ingredient of animal excretions, and if present in large amounts in water shows probable pollution with sewage. In this case the nitrates will either be high also, or entirely absent.

'Nitrogen as nitrites' indicates a changing condition of oxidation caused by the presence of bacteria, except in deep well and in rain water. The presence of measurable quantities of nitrites in river or subsoil water is considered by good authorities as sufficient ground for condemnation.

'Nitrogen as nitrates' is the final product of oxidation of nitrogenous organic matter, and if present in abundance, would indicate that the water had at some previous time been contaminatd with sewage and that bacteria have been and may still be present in the water. River and subsoil waters will always contain nitrates unless some reducing action, such as recent sewage pollution, is at work. If nitrates are higher or lower than in normal waters of the same locality and of the same class, the waters may be considered as contaminated.

'Oxygen consumed' or 'required oxygen' are terms used to indicate the amount of oxygen required to oxidize the organic carbon present. In general, a sample which will consume much oxygen will be more likely to be unwholesome than one which is low is oxygn consuming power. But the interferences are so numerous and variable that this determination is only of accessory value. It is, however, of great value for consecutive analyses of the same water supply.

'Solids on evaporation' means the total solid matter which has been dissolved or held in suspension by the water, and is determined by evaporating to dryness a measured quantity of the water and weighing the residue.

'Solids on ignition' show the solid material after the total solids have been ignited over a flame at a low red heat. Careful observation during this ignition process may reveal valuable information as to the nature of the solids.

'Loss by ignition' simply shows the measured quantity of solids driven off by the ignition.

The following may be of use in giving an idea of the permissible limits in a water for domestic use:

Free ammonia	.080 parts per million.

Albuminoid ammonia	.150 parts per million.
Chlorine	8.000 parts per million.
Nitrites	trace.
Nitrates	1.000 parts per million.
Total solids	600.000 parts per million.

As stated before, however, no set standard can be applicable to all cases. The above would be considered by most chemists as a safe basis upon which to judge a sample of water in this state, in the absence of local normals, unless a bacterial examination or the environmental survey gave conclusive evidence of contamination. The best method would be to establish 'local normals' for each separate locality in the state based upon analyses of samples from each locality known to be potable.

While the Engineering Experiment Station has not been able to take up much special investigation of the sanitary condition of waters being used for domestic purposes over the state, it has been called upon to make analyses for a large number of cities and individuals in nearly every section of the state, the results of which may be of interest. These are published in this connection without identification as to source and location except in a general way.

The first three tables given below are results of investigations made by the Experiment Station in connection with thesis work carried out by graduating students of the college.

The sanitary survey of Washington a city of 4500 population in the southeastern part of the state was made by G. T. McLean and R. M. Deming, students of the college, class of 1907, the analysis of the water samples being made in the experiment station laboratory.

The topography of the country about Washington is quite regular with a gentle slope to the south and west which is taken advantage of in the main sewage system of the city. A draw runs along its northern limit developing into a small slough which cuts through the northwestern corner of the city and eventually drains into Crooked creek. A minor draw of very little importance heads east of the C. B. & Q. railway tracks and takes a southeasterly direction Finally, the main drainage line of the city heads south of the present city wells and takes a southwesterly direction to Crooked creek, which flows around the southwest corner of the city. As a rule, the south and west sections have good drainage while the east and north portions are not so well drained.

The first water bearing stratum is limited by a blue clay at a depth of from ten to fifteen feet which extends only under the northern part of the city. A layer of sand underlies that portion of the town south of Madison street at depths of from thirty to forty feet. Typhoid has been reported from this district. A bed of sand underlies the district north of Madison street at a depth of sixty to seventy-five feet and at twenty-five to thirty-five feet water is found in red joint clay. Water bearing rock is found in the southern part of the city at from fifty to one hundred feet in what is known as the "Burlington Limestone". This limestone extends under only the southern portion of the city; the northern part lying over an old drift filled valley. In the northern part there is a water bearing layer of brown shale at two hundred feet and another in rock at from three hundred to four hundred feet. Local deposits of sand are scarce, although a bed of sand at a depth of ten to twenty feet exists in the draw through the southwestern part of the city.

The sanitary condition of Washington has in the last decade been good, according to the report of the health officer, with the exception of one instance. From 1895 to 1898 several cases of typhoid occurred each year in an isolated district south of Jefferson street and east of the C. B. & Q. railway tracks, probably due to the contamination of the surface waters of that district. This section lies on a level plain having poor surface drainage and overlying a bed of coarse sand. Since that time the cases of typhoid fever have been few, two or three a year. In 1906, cases were slightly more numerous, and in every instance the victims had used nothing but city water. It was claimed however that the disease was contracted away from the city. Very few cases of typhoid can be charged to the water and indications at present show a fairly good supply.

Sample 118 shows well oxidized matter in large amounts, which indicates that the water has been contaminated; but the source of the contamination being at some distance, good filtration is accomplished before it reaches the well. An old ceme-

TABLE NO I.

Laboratory Number....	Description and Source of Samples	Parts Per Million								
		Free Ammonia....	Albuminoid Ammonia.......	Chlorine..........	Nitrogen as Nitrites.........	Nitrogen as Nitrates........	Oxygen Consumed.	Solids on Evaporation.....	Solids on Ignition.........	Loss on Ignition..
118	Bored well. Depth to water, 18 feet. Tile casing.	0.36	.148	48.000	.000	2.400	.460	800.000	400.000	400.000
119	1200 foot drilled well. Depth to water 84 feet. City well No. 2.	.032	.068	134.000	.000	.000	3.160	2244.000	2040.000	204.000
120	235 foot drilled well. Depth to water 83 feet. City well No. 3.	.200	.228	28.000	Trace	.000	5.570	1130.000	950.000	180.000
121	1600 foot drilled well. Depth to water, 94 feet. City well No. 1.	1.180	.044	80.000	Very Slight Trace	.000	1.820	1628.000	1430.000	198.000
122	Dug well. Depth to water, 15 feet.	.024	.048	68.000	Very Slight Trace	3.000	.830	796.000	388.000	408 000
123	Dug well. Depth to water, 12 feet.	.024	.068	76.000	.000	.400	1.410	808.000	608.000	200.000
124	Dug well. Depth to water, 10 feet.	.020	.044	142.000	000	2.200	.830	1150.000	754.000	396.000
125	100 foot drilled well. Depth to water, 48 feet.	.004	.056	4.000	.000	.000	.490	422.000	344.000	78.000
129	Dug well. Depth to water, 11 feet.	.020	.002	52.000	Slight Trace	8.000	1.240	456 000	240.000	216.000
130	Bored well. Depth to water, 11 feet.	.008	.012	20.000	Slight Trace	4.000	1.240	212.000	146.000	66.000

TABLE NO. II.

Laboratory Number..	Description and Source of Samples.	Parts Per Million								
		Free Ammonia....	Albuminoid Ammonia.......	Chlorine..........	Nitrogen as Nitrites.........	Nitrogen as Nitrates........	Oxygen Consumed.	Solids on Evaporation.....	Solids on Ignition.........	Loss on Ignition..
143	35 foot driven well. Depth to water 6 to 8 feet. Privy vault 25 feet away. Another 35 feet away. Manure in hole 3 or 4 feet deep around well.	.004	.012	32.000	.000	2.800	.415	626.000	506.000	120 000
144	18 foot driven well. Privy vault 20 feet away.	.028	.024	28.000	Trace	.600	.831	734.000	622.000	112.000
145	30 foot driven well. Stable and privy 50 feet away.	.020	.016	28.000	.000	2.400	.415	716.000	536.000	180.000
209	16 or 18 foot driven well. 75 feet from barn. Condition bad. Standing water. Chicken yard near.	.002	040	92.000	.000	4.800	.000	928.000	676.000	252.000
210	15 or 20 foot driven well. Barn and privy 70 feet. Low ground. Water and manure all over lot.	.020	.128	40.000	Very Slight	8.000	.000	534 000	386.000	148.000
146	26 foot driven well on bottom land. Privy vault 30 feet away.	.008	.004	40.000	.000	3.600	.499	966.000	660.000	306.000
147	20 foot open dug well. Well up in the hills.	.008	.044	10.000	.000	2.000	3.326	348.000	228.000	120.000
148	24 foot driven well in lower end of draw, which may drain a large part of the town. Barn and privy vault 100 feet away.	.008	.024	52.000	.000	4.000	.831	964.000	616.000	348.000

TABLE NO. II. (Continued)

Laboratory Number.....	Description and Source of Samples	Parts Per Million								
		Free Ammonia....	Albuminoid Ammonia.......	Chlorine..........	Nitrogen as Nitrites.........	Nitrogen as Nitrates........	Oxygen Consumed.	Solids on Evaporation.....	Solids on Ignition.........	Loss on Ignition..
211	80 foot dug well. Barn 10 feet away. Oak buckets. Rusty chain. Muddy rope.	.040	.320	20.000	.000	1.600	3.575	730.000	578.000	152 000
212	20 foot dug well. Water hear surface. In bottom of draw.	016	.076	36.000	Trace	16.000	.415	960.000	580.000	380.000
213	City water. From sink tap. Nearest to a dead end. Said to give bad odor if not flushed every few weeks.	.004	.048	14.000	.000	2.000	.249	450.000	362.000	88.000
214	23 foot dug well. Two cesspools and privy 70 feet away. Standing water in lot. Filth in alley.	.120	.032	60.000	Trace	8.000	.332	852.000	552.000	300.000
215	100 foot drilled well.	.380	.076	4.000	.000	.000	.249	380 000	240.000	140.000
216	24 foot dug well in a draw (used for 5 families.) Standing water in ditch near.	.048	.056	104 000	.000	32.000	.415	1406.000	850.000	556.000
194	125 foot drilled well. Depth to water 65 feet. Barn 15 feet away; privy 10 feet away.	.700	.032	2.000	Trace	.240	1.247	406.000	338.000	68.000
195	16 foot driven well. Barn 60 feet away.	.000	.024	13.000	.000	.160	.665	444.000	306.000	138.000
196	Well dug 12 feet, bored 14 feet. Privy 35 feet away; Mud creek 35 feet away.	.048	.176	66.000	.000	.320	2.494	1094.000	750.000	344 000
197	10 foot dug well. Depth to water 7 feet Privy vault 30 feet away.	028	.088	40.000	.000	.400	1.663	828.000	542.000	286.000

TABLE NO. II. (Concluded)

Laboratory Number	Description and Source of Samples.	Parts Per Million								
		Free Ammonia	Albuminoid Ammonia	Chlorine	Nitrogen as Nitrites	Nitrogen as Nitrates	Oxygen Consumed.	Solids on Evaporation	Solids on Ignition	Loss on Ignition
198	26 foot driven well. Barn 50 feet away. Privy vault 27 feet away.	.002	.020	5.000	.000	3.200	1.164	392.000	340.000	52.000
199	28 foot driven well. Privy 50 feet away.	1.508	.040	2.000	.000	.080	2.827	396.000	326.000	70.000
200	33 foot dug well. Barn 60 feet away. Privy vault 40 feet away.	.048	164	53.000	.000	.240	.997	1046 000	624.000	422.000
201	52 foot driven well at city pumping station.	.008	.024	12.000	Very Slight	.080	1.164	466.000	366.000	100.000

In sample 215 the free ammonia is very high, but as the chlorine is quite low and the nitrites and nitrates are entirely absent, the water would not be considered bad, coming as it does from such a deep well which is well cased and protected from surface drainage by a gentle slope from the well in all directions.

From the table of analysis it will be noted that the best water is furnished by the deep wells on the hills, but these can not all be considered absolutely safe. The majority of wells show sewage contamination in process or in complete state of purification. The wells located along the lower part of Mud creek, which forms a distributing system for the sewage of a wide district, are especially dangerous. The safety in using many of the best waters depends upon the power of the soil and natural agencies to continue their work of purification. In many cases only the introduction of disease bacteria is needed to cause an epidemic.

In a city where so much of the water supply is from private wells, and where the sewage disposal is entrusted so completely to nature, the sanitary condition can not be good, and a sewerage system is certainly much needed. The city wells located near the center of the city draw their supply of water from the bed of gravel to which doubtless a large part of the city's sewage sinks. If the sewage is not to be properly taken care of, the water supply should be brought from a source far enough away to preclude any possible contamination.

Audubon, a city in the western part of the state, was surveyed by C. D. Forsbeck, Class '08, and examination made of a number of samples of water being used. Audubon is a small city of about two thousand inhabitants. The general health conditions at the time this investigation was made were very good and very few cases of typhoid have been reported in the past.

The topography here is somewhat rolling. The general trend of the slope is to the southwest. A small creek runs through the west part and several ravines lead back through the east portion of the city. No high hills are found, and the difference of elevation between the highest and lowest points does not exceed one hundred and fifty feet. The natural drainage is fair. The soil is of the Kansan drift and varies within short distances. For example, one well was dug and under the black soil nothing but dry yellow clay was found for twenty five feet, with no water. Another well was sunk ten feet away and within twenty-five feet from the surface yellow clay, white sand, and some gravel were encountered with plenty of water, which, however, disappeared within a few months. Below this surface wash a clay is found, varying in color from black to a dark brown. This dark blue clay extends to a depth of over three hundred feet. Much vegetable matter is found in this clay, and the water from some wells which penetrate it gives off very offensive odors due doubtless to the decomposition of the organic matter contained in it.

The water used in the city comes from shallow wells, and is found just above the blue clay in a thin stratum of sand at depths ranging from fifteen to twenty-five feet in the bottom to fifty feet on the hills. The bottom of the city well is four feet in the blue clay, and in it were found leaves, wood, and other material of a vegetable nature.

The sanitary conditions can not be perfect in a town of this size where no sewer system is maintained and the drainage is left to nature. Nearly every residence has a well and privy vault on the same lot; a few cess-pools are scattered throughout the city. Barns and manure piles are numerous. The city wells are located along the creek, and the survey shows that the east and north parts of the city drain toward these wells. A good deal of the water is seepage, but at a depth of thirty feet a vein is found coming from a northeasterly direction. The city well is thirty-five feet deep and twenty-five feet in diameter. The supply is said to be satisfactory.

The analyses of samples of water collected are shown in Table III.

A study of sample No. 430 shows the free ammonia high and very much higher than the albuminoid ammonia, which is an objectionable indication. The chlorine is abnormal, and the presence of both nitrites and nitrates would indicate that sewage was present in an unpurified condition.

The excess of chlorine and nitrates in sample No. 431 indicates that sewage has been introduced after the nitrogenous matter was well oxidized.

Sample No. 432 is from a dug well about thirty-two feet deep which is used by about twenty families. A barn and privy

TABLE NO. III.

Laboratory Number	Description and Source of Samples.	Parts Per Million Free Ammonia	Albuminoid Ammonia	Chlorine	Nitrogen as Nitrites	Nitrogen as Nitrates	Oxygen Consumed	Solids on Evaporation	Solids on Ignition	Loss on Ignition
430	Dug well. Depth to water, 10 feet.	.116	.036	48.000	Slight Trace	6.400	.249	736.000	534.000	202.000
431	Bored well. Depth to water, 20 feet. Barn and outhouse 12 feet away.	.016	.052	292.000	Slight Trace	32.000	1.081	1870.000	1192.000	678.000
432	Bored well. Depth to water, 25 feet. Barn and outhouse 50 feet away.	.024	.064	120.000	Trace	12 800	.665	1350.000	1000.000	350 000
433	Bored well. Depth to water, 45 feet. Outhouse 75 feet away.	.004	.028	16.000	.000	1.600	.199	514.000	440.000	74.000
434	Bored well. Depth to water, 20 feet. Barn 60 feet away.	.000	.008	4.000	.000	.800	.099	336.000	288.000	48.000
435	Bored well. Depth to water, 25 feet. Outhouse and barn 75 feet away.	.028	.056	12.000	Slight Trace	22 400	.183	560.000	318.000	242.000
443	Bored well. Depth to water, 15 feet. Outhouse and barn 80 feet away.	.068	.064	12.000	.000	.000	1.845	284 000	234.000	50.000
444	Bored well. Depth to water, 18 feet. Barn and outhouse 60 feet away.	.028	.084	6.000	Very Slight Trace	.800	.814	374.000	280.000	94.000
445	Bored well. Depth to water, 15 feet. Barn 60 feet away; outhouse 40 feet away.	080	.064	35.000	Very Slight Trace	.240	1.538	1200.000	984.000	216.000
446	Bored well. Depth to water 16 feet. Cesspool 300 feet away. Ground drains toward this well. (Wooden pump.)	008	.032	24.000	Slight Trace	.550	.582	476.000	364.000	112.000
447	Bored well. Depth to water, 20 feet. (Wooden pump.) Cesspool 200 feet away. Outhouse 100 feet away;.	.052	.116	147.000	Trace	1.280	.914	1580.000	1058.000	522.000
448	Bored well. Depth to water 25 feet. Outhouse and barn 70 feet away.	.176	.056	32.000	Trace	1.120	1.164	782.000	612.000	170.000

With one or two exceptions the analyses of the water from wells in this town shows them to be badly polluted.

vault within fifty feet of the well probably furnish the pollution which is indicated by the presence of much chlorine and nitrates. The presence of nitrates shows that the nitrogenous matter is still in a changing condition.

Sample No. 433 would not be considered by most chemists as a safe water.

No. 434 is a very good water as indicated by the sanitary analysis.

The results of the investigations make it evident that much of the water used is in very bad condition, due largely to failure to properly take care of the sewage. There is abundant opportunity for the introduction and development of disease germs which can only lead to an epidemic at any time. The installation of a sanitary sewerage system would be advisable.

Table Number IV includes a number of miscellaneous analysis of samples sent to our laboratory by superintendents of water works in various cities and towns and by city engineers and private individuals where sanitary conditions have been under suspicion.

Unless vertical cracks or fissures in the strata exist, the water from drilled wells 100 or more feet in depth is usually safe for domestic use. Conditions, however, may often be favorable for the contamination of wells several hundred feet deep. For example, the stratum in which the water is found may have its outcrop in a city located some distance away from which it receives the vilest kind of pollution and carries it with little change to the supply source. All open and shallow wells are very dangerous as they are subject to contamination at any time.

Even though a chemical analysis of water from such sources may show it to be very good at the time the sample was taken. there is great danger in placing too much reliance upon the analysis and disregarding the serious possibility of pollution to which wells of this class are continually subjected.

The sources of contamination will not, as a rule, all be removed by the sanitary sewage disposal plant, as there will always be more or less decomposing garbage and refuse material from barns and from alleys and streets which may be washed into open wells by heavy rains or dissolved and car-

TABLE NO. IV.

Laboratory Number....	Description and Source of Samples.	Parts Per Million								
		Free Ammonia....	Albuminoid Ammonia.......	Chlorine..........	Nitrogen as Nitrites.........	Nitrogen as Nitrates........	Oxygen Consumed.	Solids on Evaporation.....	Solids on Ignition.........	Loss on Ignition..
7	This group of eleven analyses is of various wells located in different parts of a certain town in western Iowa. The tests were made in an investigation for the purpose of aiding in the location of a desirable water supply for the town.	.024	.048	50.000	High	10.000	1.420	1810.000	1200.000	610.000
8		.012	.044	36.000	Heavy Trace	6.000	1.180	1302.000	904.000	398.000
9		016	.052	20.000	Trace	4.000	2.700	622.000	442.000	180 000
10		.008	.004	394.000	.000	10.000	1.020	2722.000	1566.000	1156.000
11		.004	.008	5.000	Slight Trace	1.200	.540	482.000	296.000	186.000
25		.040	.188	4.000	High Trace	.200	5.760	480.000	340.000	140.000
26		.044	.460	5.000	Trace	.000	19.092	704 000	515.000	189.000
49		.060	.288	20.000	Trace	1.600	4.360	560.000	400.000	160.000
50		.040	.568	20.000	.000	.000	14.000	996.000	622.000	374.000
91		.020	.020	10.000	.000	.000	3.335	440.000	380.000	60.000
92		.040	.036	10.000	.000	Slight Trace	2.501	452.000	390.000	62 000
12	A city supply. Tap water. Reservior is filled from both artesian and shallow water.	.440	.040	6.000	Trace	.200	.836	394.000	300.000	94.000
x x	Drilled well about 95 feet deep.	1.580	.232	5.000	.000	.000	5.460	434.000	286.000	148.000
x x 54x	Drilled well about 100 feet deep, southeastern Iowa.	.054	.060	5.000	.000	.000	.000	430.000	346.000	84.000

TABLE IV. (Concluded).

Laboratory Number	Description and Source of Samples	Parts Per Million								
		Free Ammonia	Albuminoid Ammonia	Chlorine	Nitrogen as Nitrites	Nitrogen as Nitrates	Oxygen Consumed	Solids on Evaporation	Solids on Ignition	Loss on Ignition
55x	Dug well. Depth to water, 6 feet. Typhoid in vicinity. Outhouse 40 feet away. Refuse near well.	.034	.060	66.000	.000	15.000	Trace	1036.000	656.000	380.000
270	Shallow well in low ravine.	.020	.100	44 000	Heavy Trace	4.040	.831	1090.000	846.000	244.000
284	Shallow driven well, northern Iowa.	.840	.080	8.000	Trace	Trace	.000	574.000	446.000	128.000
285	Spring water. Old rotten wood and rotten straw, etc., all about the spring.	.260	.044	30.000	.000	.000	.000	744.000	534.000	210 000
293	A new spring dug a little above the old one, (No. 285.) protected from cattle and surface contamination.	.164	.092	2 000	Slight Trace	.000	1 750	516.000	458.000	58.000
327	100 foot drilled well. Well protected; central Iowa.	1.260	.104	3.000	.000	.000	1.912	374.000	338.000	36.000
328	Water from an Iowa lake.	.020	.520	2.000	.000	.000	5.487	244.000	150.000	94.000
329	60 foot drilled well, northwestern Iowa, near a pond of standing water.	.212	.130	135.000	.000	.000	4.157	1322.000	938.000	384.000
442	Drilled well in western Iowa.	.164	.032	7.000	.000	Trace	.498	376.000	326.000	50.000
737	Artesian well in central Iowa.	1.800	.032	1.000	Trace	.000	972	482.000	392.000	90.000
777	Shallow driven well. Same locality as No. 284.	.688	.040	7.000	Trace	.800	1.496	570.000	476.000	94.000

The best water represented in any of the above tables is shown by the analysis of sample number 54x. But by far too many well of the type represented by numbers 329, 270 and 55x and others, are in use all over the state.

There are doubtless many good wells in the state of Iowa, and many which are only fairly good, that would be greatly improved if protected from sewage pollution.

ried through the soil unpurified to the bed from which the shallow well is supplied.

The modern well drilling machines which are to be found in nearly every community, should place the possibility of a deep well supply within reach of every city at reasonable cost.

In considering the sanitary examination of waters however briefly, it should be appreciated that chemical investigations are but one phase of the question; and that in order to gain complete and conclusive knowledge of the value of a water, they should be accompanied by a bacteriological examination. This is of very great importance, especially with waters from sources that are in any way subject to the possibility of surface contámination. When all the figures in the sanitary analysis are high, or when certain figures are extremely high, the chemist can consistently condemn the water under investigation. It does not follow, however, that a water in which all the values are low is perfectly healthful, as the chemical tests alone might indicate; but the presence of pathogenic bacteria in the water might render it far from sanitary or even dangerous to health and yet their presence not even be suggested by the chemical results. In order to be absolutely certain as to the potability of water, the sanitary chemical analysis, even though the results are very low, should be supported by a bacteriological examination. These together with a careful environmental survey of the source of supply should furnish very reliable data, providing, of course, that the samples are properly collected and placed in proper receptacles for shipment to the laboratory.

The methods used in making the above analyses are selected from those recommended by a committee on Standard Methods of Water Analysis to the Laboratory Section of the American Public Health Association in 1905.

Instructions for collecting and shipping samples of water for analysis will be mailed to any desiring to have their water supply investigated. For such tests, in the case of residents of the state, the charge covers only the bare cost of the work.

BULLETINS OF THE ENGINEERING EXPERIMENT STATION

(Copies may be had by addressing A. Marston, Director, Ames, Iowa.)

No. 1.—The Iowa State College Sewage Disposal Plant and Investigations.
No. 2. Bacteriological Investigations of the Iowa State College Sewage.
No. 3. Data of Iowa Sewage and Sewage Disposal.
No. 4. Bacteriological Investigations of the Iowa State College Sewage Disposal Plant.
No. 5. The Chemical Composition of the Sewage of the Iowa State College Sewage Disposal Plant.
No. 6. Tests of Iowa Common Brick.
No. 7. Sewage Disposal in Iowa.
No. 8. Tests of Dry Press Brick Used in Iowa.
No. 9. Notes on Steam Generation with Iowa Coal.
No. 10. Dredging by the Hydraulic Method.
No. 11. An Investigation of Some Iowa Sewage Disposal Systems.
Vol. II. No. 6. The Good Roads Problem in Iowa.
Vol. III. No. 1. Tests of Cement.
Vol. III. No. 2. State Railroad Taxation.
Vol. III. No. 3. Steam Generation With Iowa Coals.
Vol. III. No. 4. Incandescent Lamp Testing.
Vol. III. No. 5. Steam Pipe Covering Tests.
Vol. III. No. 6. The Assessment of Drainage Districts.
Vol. IV. No. 1. Tests of Iowa Limes.
Vol. IV. No. 2. Holding Power of Nails in Single Shear.
Vol. IV. No. 3. Effects of Coloring and Water Proofing on the Strength and Permeability of Cement Blocks.—The Improvement of Cement Mortar by Grading the Size of the Sand.—Tests of the Impermeability of Reinforced Concrete Pipes.
Vol. IV. No. 4. Miracle Prize Papers for 1909.
Vol. IV. No. 5. Sanitary Examination of Well Water.
Results of Drainage Investigations for 1908 and 1909. (In Preparation.)
Tests and other Investigations of Cement Drain Tile and Sewer Pipe. (In Preparation.)

4

Engineering Experiment Station

IOWA STATE COLLEGE

Ames, Iowa

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VOL. IV BULLETIN NO. VI

Sewage Disposal Plants for Private Houses

BY

A. MARSTON AND F. M. OKEY

DECEMBER, 1909

THE
TORCH
PRESS
CEDAR RAPIDS
IOWA

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SEWAGE DISPOSAL PLANTS FOR PRIVATE HOUSES

By A. Marston and F. M. Okey.

Ever since its establishment, the Engineering Experiment Station has been receiving a continuous stream of inquiries for information concerning suitable sewage disposal plants for private houses. Hence, one of the first lines of investigation undertaken by the station was the designing, construction and operation of such plants.

The work of investigation has required much more time than was at first anticipated. To secure conclusive information as to the operation of any especial plant, it has been necessary to operate it at least a year under actual working conditions. Many unexpected difficulties were encountered in connection with the development of a successful plant, and each difficulty developed by a new plant required another year of work to overcome. Hence, although our work was begun in 1904, we have not been willing to publish the results in bulletin form until now, and even yet we do not feel that our work is by any means concluded. Investigations will be continued, probably for several years to come, and the right is reserved to change our views with continued experience.

DIFFERENCE IN SEWAGE DISPOSAL CONDITIONS OF PLANTS FOR PRIVATE HOUSES AND FOR CITIES.

In attacking this problem use was naturally made of the experience of engineers in the design and operation of sewage disposal plants for cities. The first plants were designed along the same lines as the large ones required for towns, with simplified details. Early in the work, however, it became apparent that there were many difficulties peculiar to small plants as distinguished from large. The engineer who expects to secure results per unit of area in the case of sewage disposal plants for private houses equal to those obtained in the case of large sewage disposal plants is certain to be badly disappointed. Some of the points of difference are as follows:

1. *Extreme fluctuations in flow.* The sewage of a city consists of a continuous stream of considerable volume, which, it is true, fluctuates within considerable limits, but which is very much more steady than the flow from any one of the several houses. In the case of the latter, there is a great rush of sewage when any one fixture is discharged, but during much or even most of the time there is no flow at all. In the case of the individual house, the fluctuations of flow are enormously greater in proportion to the size of the tank and area of filter beds than in the case of the city plant.

2. *Small size of sewage tank.* In the case of the city plant, the size of the tank required is so very large that the fluctuations in the rate of flow do not, if the tank is well designed, stir up the deposits and carry them out upon the filters. In the case of the individual house, however, the tank is of very small dimensions. The sudden inrush of sewage at the time of the discharge of any fixture stirs up the entire contents of the tank, disturbing the bacterial conditions and carrying out the suspended matter upon the filtering area, thereby causing it to clog. Several times the area of filter needed in city plants per person will be required in the case of the private sewage disposal plant.

3. *Fresh character of the sewage from the individual house and fluctuations in quality.* In the case of the city plant, the sewage has been carried long distances in the sewers. The suspended matter has been macerated and finely divided by mechanical agencies during the flow, and bacterial action has started in the sewers, and in many cases has proceeded so far as to effect a material change in the condition of the sewage. The mingling of the sewage from many houses gives a comparatively uniform composition, although even in the case of city plants the composition of the sewage fluctuates widely.

The sewage from the private house, however, is received directly at the plant, without mechanical maceration or division of the suspended matter, and without bacterial change. Its composition varies from solid fecal matter to pure water. Obviously, a much larger tank in proportion will be required in the case of the private disposal plant than in the case of the city plant, in order to provide for the different conditions.

4. *Impossibility of securing proper maintenance of private sewage disposal plants.* Even in the case of a city with a plant so large that it should receive constant care, it is very difficult to secure proper maintenance. This is reported by all engineers. Most of the failures in sewage disposal plants are due to neglect of the plants on the part of the cities owning them. However, this condition is one which is capable of being remedied on the part of cities, and there is a movement for better conditions. Sanitary engineers are giving especial attention to the subject. The expensive teachings of experience are proving to the cities that they must do better in this particular.

In the case of private plants, however, it is absolutely hopeless to secure other than the most casual care of the plant. The ordinary citizen cannot be expected to possess the knowledge necessary to give scientific care to a sewage disposal plant. It is absolutely certain that he would not give it such care even if he possessed the knowledge. The private plant, to be successful, must be capable of operating at least a year with practically no attention whatever.

5. *Near proximity of private plants to dwelling houses.* In the case of the city plant, more or less odor is expected, and provision is made against its being a nuisance by constructing the plant at a considerable distance from the nearest dwelling house. In many if not most cases this would be impracticable in the case of the private plant. It must be built, often, within the confines of the same single lot on which the dwelling is placed. It must, therefore, be practically free from objectionable odor.

The authors of this bulletin have read many articles on sewage disposal plants for private houses. In these articles we have not seen proper attention given to the above differences between such plants and those for cities. If the writers of these articles would actually construct their plants and watch their operation closely under actual working conditions for a year or more it would be found that their ideas would be materially modified by the difficulties encountered, due to the points of difference enumerated above.

THEORY OF SEWAGE PURIFICATION

To understand our reasons for the principal features of the different sewage disposal plants experimented with and the difficulties encountered in connection with each, some knowledge of the general theory of sewage purification is necessary. While there are peculiar difficulties to be overcome in the purification of sewage in private disposal plants, the general principles of purification are the same as in the case of large plants for cities.

In modern sewage purification, the process is partly mechanical and partly bacterial. While of late years most attention has been given to bacterial agencies of purification, engineers are now coming to realize that close attention must also be given to the mechanical principles involved.

In modern plants, the process of purification is nearly always divided into two parts; first, *preliminary treatment* (or tank treatment); second, *final treatment* (or filtration).

In *sewage tanks*, the aim is to separate the suspended matter in the sewage from the liquid part, and so far as possible to destroy it by bacterial agencies. The modern sewage tank is nothing but a magnified and improved cesspool. It is interesting to recall that the origin of all sewage purification methods was the private sewage disposal plant, usually, heretofore, consisting simply of a cesspool. From time immemorial, cesspools have been constructed, and we now know that bacterial agencies of purification do active work in such pools of sewage.

A sewage tank, therefore, preferably forms a part of a private as well as of a city sewage disposal plant. The bacteria working therein are principally those which can exist without the presence of oxygen. They, therefore, belong to one of the two classes, known to bacteriologists as "*anaerobic*," or those which do not require oxygen to live, and "*facultative*," or those which can live either with or without the presence of oxygen.

In a sewage tank the facultative and anaerobic bacteria use as food the organic matter which has settled out of the sewage in the form of sludge, and thereby partially destroy it. In this partial destruction, a portion of the sludge not absolutely destroyed is liquefied. To a very limited extent, the bacteria in the tank also have the power of feeding upon and partially destroying organic matter in solution in the sewage.

To make the conditions most favorable for the action of the bacteria, the sewage in the tank should be entirely at rest and light and air should be excluded.

However, at the very best, the total amount of purification which can be effected by bacteria in the sewage tank is comparatively small. In city plants, the amount usually ranges from 25 to 50 per cent of purification. The effluent from such tanks is still a foul and dangerous liquid, swarming with sewage germs. It would be dangerous and cause a nuisance if discharged into tile drains or ditches without further purification.

To carry on the process of purification further, until it reaches a point which will afford a safe effluent, it is necessary to add *filtration.* In city plants, two general types of filters are in use at the present time; first *sand filters*, and second, *sprinkling filters.*

Sand filters are made of ordinary coarse, clean, mortar sand. For their satisfactory operation, the sewage must be applied to them in intermittent doses, and the surface of the sand kept in such loose and clean condition that the sewage will disappear entirely within a very short time after application of each dose of the sewage. Sand filters will not work satisfactorily when the surface is kept flooded, as under these conditions the aerobic bacteria of purification cannot exist.

In sewage filters, the purification is effected by the agency of "*aerobic*" bacteria, or those which require oxygen to live. These germs live in uncounted billions in a slimy bacterial coating covering the surface of the filter grains. In the case of sand filters, the purification germs do the most of their work at the surface of the filter.

When a sand filter is constantly kept flooded with sewage the aerobic bacteria cannot live, because they have no means of getting oxygen. By applying the sewage in intermittent doses which disappear after the application of each dose, air is drawn down into the pores of the sand as the sewage disappears and oxygen is supplied to keep the bacteria of purification alive.

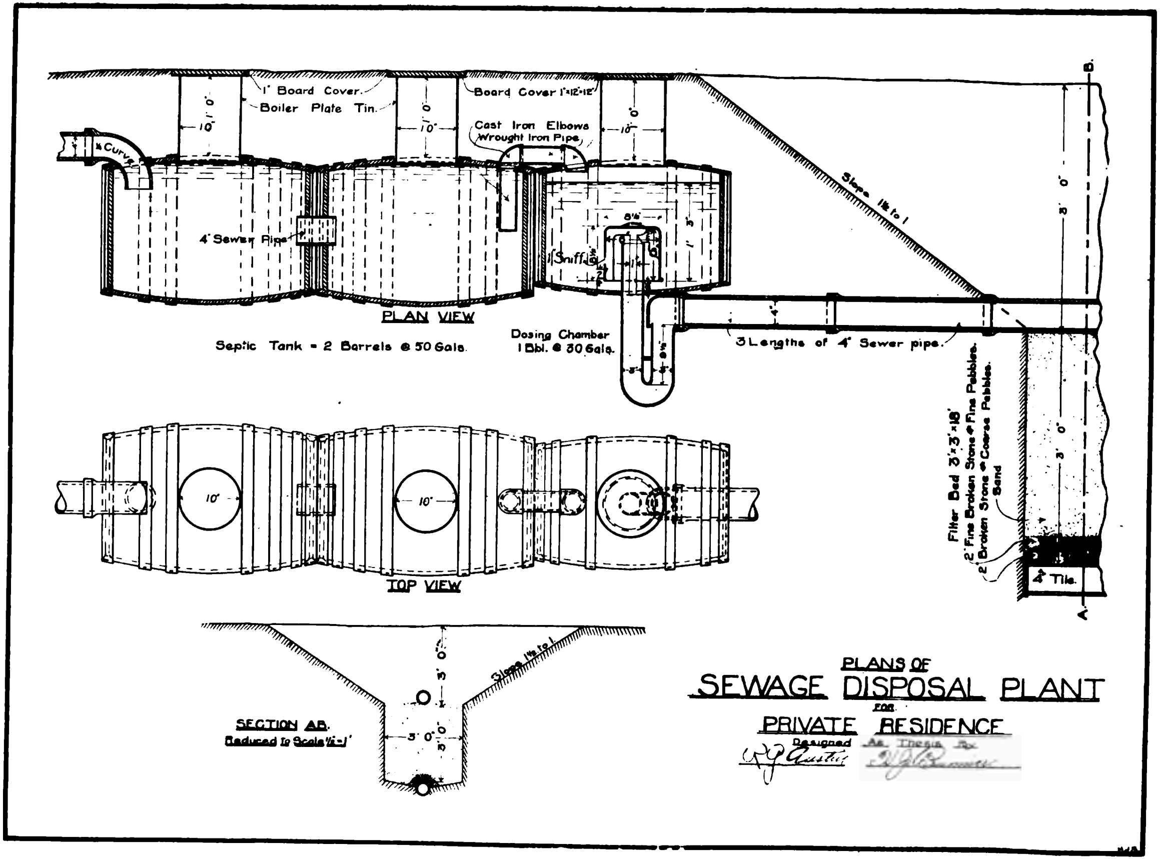

1" Board Cover.
Boiler Plate Tin.
Board Cover 1"×12"×12"
Cast Iron Elbows
Wrought Iron Pipe
¼ Curve
10"
1' 0"
4" Sewer Pipe
Sniffle
8½"
1' 3"
PLAN VIEW
Septic Tank = 2 Barrels @ 50 Gals.
Dosing Chamber
1 Bbl. @ 30 Gals.
3 Lengths of 4" Sewer pipe.
Slope 1½ to 1
3' 0"
B
A
Filter Bed 3'×3'×18'
2" Fine Broken Stone & Fine Pebbles.
2" Broken Stone & Coarse Pebbles.
Sand
4" Tile.
TOP VIEW
10"
SECTION AB.
Reduced to Scale ½"=1'
Slope 1½ to 1
3' 0"
PLANS OF
SEWAGE DISPOSAL PLANT
FOR
PRIVATE RESIDENCE
Designed
As Thesis for

FIGURE 1

EXPLANATION OF FIGURE 1.

Figure 1. shows the first private sewage disposal plant experimented with. It was constructed in 1904 and proved unsatisfactory, because of the failure of the siphon to operate properly, and because of the rapid clogging of the sand filter.

Hence, sand filters require constant care to maintain the surface in a loose and clean condition.

The other type of sewage filters, known as *sprinkling filters*, is made of coarser material than sand. Usually pebbles screened free of sand, or crushed stone are used, the size used in city plants being usually from ¼ inch to 2 inches. This coarse material has air passages throughout its extent, of sufficient size to permit the circulation of air when the sewage is trickling through, and hence the aerobic bacteria can be kept alive even when the sewage is applied continuously. However, it is necessary to have the sewage sprinkled over the surface in drops, as, otherwise, it would flow through this coarse material so quickly as hardly to come in contact with the bacteria at all, and it would fill the pores of the material so as to exclude air and destroy the bacteria of purification. In such plants, the sewage is usually sprayed over the surface from nozzles, very much as water is sprayed over the lawn in the summer time. In passing through the air, the drops of sewage are freed from objectionable gases and take up oxygen.

To recapitulate, then, the general theory of sewage purification is as follows:

In *sewage tanks* the solid matter in the sewage is allowed to settle out and is partially destroyed and liquefied by "*facultative*" and "*anaerobic*" bacteria, which also slightly affect the dissolved organic matter. The effluent from the tank is then filtered through *sewage filters*, in which "*aerobic*" bacteria carry on further the process of sewage purification. It is necessary to supply these aerobic bacteria with air. The aerobic bacteria effect a much greater degree of purification than the anaerobic and the facultative.

It may be added that the effluent of a good sand filter, properly cared for, will be as clear and odorless as spring water, and will have a very small percentage of bacteria left in it. The effluent from sprinkling filters will not be nearly so good in appearance as that of sand filters, and will have a much higher percentage of germs remaining, but if the sprinkling filters work properly, the effluent therefrom will be purified to such an extent as not to be subject to further putrefaction.

EXPERIENCE WITH EXPERIMENTAL PLANT NO. 1.

Experimental plant No. 1 is shown in Figure 1. This plant was constructed in 1904. As is shown by the figure, the sewage tank consisted of three barrels placed end to end and connected by iron pipe. The third barrel served as a dosing chamber to afford intermittent application of the sewage to the sand filter. In this third barrel was placed a 3 inch Miller sewage siphon. The operation of this siphon is such that when the dosing chamber fills to high water line the siphon will suddenly empty it. The flow from the siphon then ceases and the chamber refills.

The sand filter in this case consisted of a trench 3 ft. wide and 3 ft. deep and 18 ft. long, filled with sand and pebbles to a depth of 3 ft. overlying a 4 inch tile drain.

The use of the barrels was for the reason that this was an experimental plant only, and the cheapest temporary construction possible was considered sufficient to determine whether the plant would or would not work satisfactorily.

In operation, this plant was not found to work satisfactorily. The rushes of sewage from the house sewer thoroughly stirred up the contents of the barrels and carried out much sediment to the filter bed, which caused a quick clogging thereof. In an attempt to remedy this, the size of the filter was largely increased but without overcoming the difficulty.

Another difficulty which was not overcome was the uncertainty of action of the siphon. It was found that to insure the proper working of such a siphon would require regular examination and attention, such as could not reasonably be expected in the case of a private sewage disposal plant.

For the above reasons, the plant was considered to be a failure, in that it did not constitute a satisfactory working system for a private sewage disposal plant. However, it developed quite clearly many of the unexpected and peculiar difficulties in private disposal plants as compared with city plants, which have already been enumerated.

Plant No. 1 was designed, under the direction of one of the writers of this bulletin, by Messrs. R. J. Austin and H. J. Brunnier, seniors in the Civil Engineering Department, for their thesis. They also constructed the plant and watched its operation until their graduation a few months later.

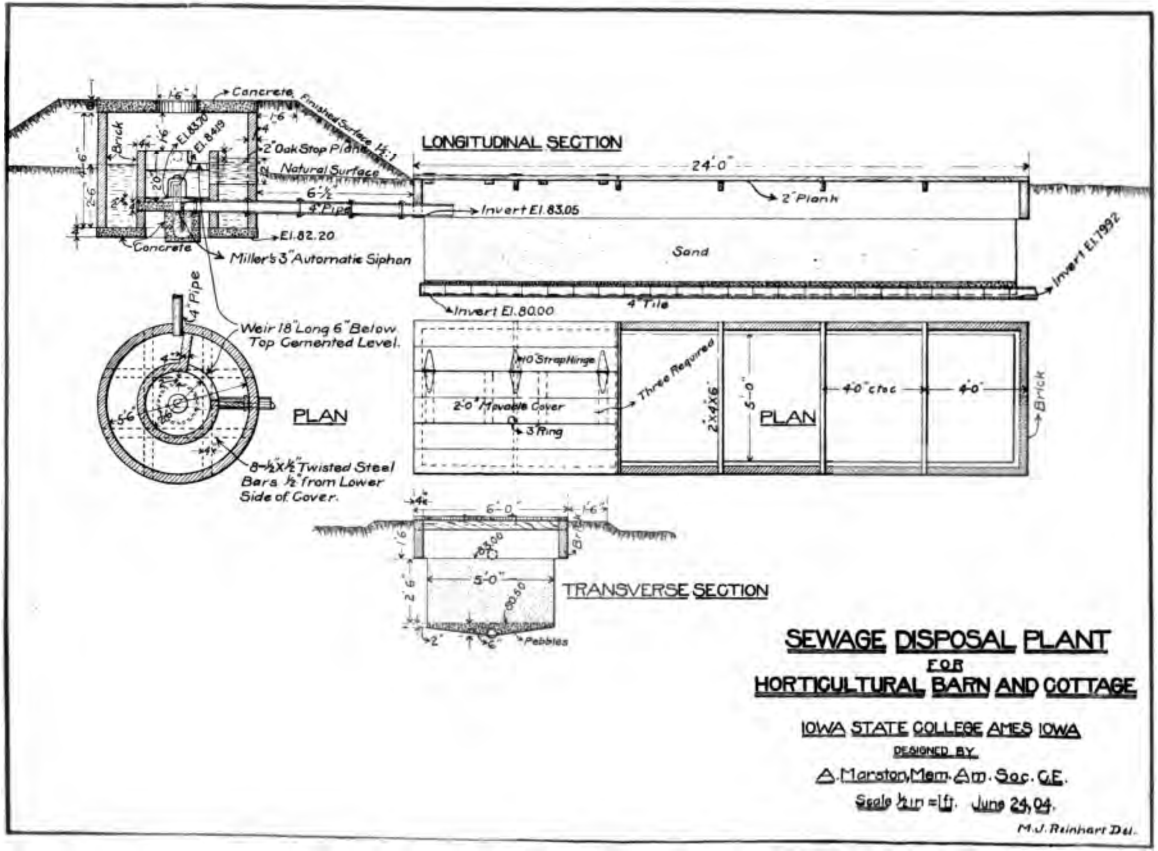

LONGITUDINAL SECTION
24'-0"
2" Plank
Invert El. 83.05
Sand
4" Tile
Invert El. 80.00
Invert El. 79.92
Concrete
Finished Surface 1½:1
2" Oak Stop Plank
Natural Surface
6'-½"
4" Pipe
El. 82.20
El. 83.70
El. 84.19
Brick
Concrete
Miller's 3" Automatic Siphon
4" Pipe
Weir 18" Long 6" Below Top Cemented Level.
PLAN
8-½"x½" Twisted Steel Bars ½" from Lower Side of Cover.
10" Strap Hinge
2'-0" Movable Cover
3" Ring
Three Required
2"x4"x6"
5'-0"
4'-0" c to c
4'-0"
PLAN
Brick
6'-0"
1'-6"
2'-6"
5'-0"
83.00
80.50
Pebbles
TRANSVERSE SECTION
SEWAGE DISPOSAL PLANT
FOR
HORTICULTURAL BARN AND COTTAGE
IOWA STATE COLLEGE AMES IOWA
DESIGNED BY
A. Marston, Mem. Am. Soc. C.E.
Scale ½ in = 1 ft. June 24, 04.
M. J. Reinhart Del.

FIGURE 2

EXPLANATION OF FIGURE 2.

Figure 2 shows the second private sewage disposal plant experimented with. The design of the tank is such as to obviate all flow through small pipes, and it was hoped that less sediment would be carried over upon the filter beds. The sand filter, too, was made much larger than in plant No. 1. This plant was constructed in 1904 and has been in continuous operation ever since. In operation, the siphon does not work satisfactorily, the sand bed soon became clogged and remains constantly flooded with sewage to a considerable depth. The wooden cover to the sand filter has decayed. The effluent is comparatively foul, but has run off in an ordinary tile drain without causing stoppage so far.

On the whole, the plant is not considered satisfactory.

EXPERIENCE WITH EXPERIMENTAL PLANT NO. 2.

After some months of experience with experimental plant No. 1, it became necessary to construct a plant to purify the sewage from the college Horticultural Cottage and Barn. In view of the experience with Plant No. 1, a design was prepared which is shown in Figure 2.

The sewage tank was circular in form, as shown in Figure 2, and was constructed entirely of brick and concrete. The dosing chamber was also circular and occupied the center of the main tank. The sewage entering was compelled to flow entirely around this circular dosing chamber before entering it. For entrance to the dosing chamber, a weir 18 inches long was provided, to reach which it was necessary for the sewage to pass under a wooden baffle-board. It was hoped by this means to prevent disturbance of the sediment in the tank and consequent early clogging of the filter beds.

The siphon provided in the dosing chamber was the same as that in Plant No. 1, i. e., a 3 inch Miller siphon. In actual operation, it was found not to work satisfactorily without comparatively frequent examination and care.

The sand filter bed was 5 ft. wide by 23 ft. long in the clear, and contained approximately twice as many square feet as in the case of Plant No. 1. This filter bed was enclosed to protect it from the weather.

In operation, it was found that the sand beds, without more regular care than could be expected for a private sewage disposal plant, would soon become clogged with sewage. This and the failure of the siphon to work satisfactorily without especial care have made this plant unsatisfactory to its designers. However, it must be admitted that it disposed of the sewage from the Horticultural Cottage and Barn satisfactorily from the date it was installed until the fall of 1909. At that time, complaint was made by the occupants of the cottage, which complaint seems to have been based more upon the decay of the wooden construction and consequent exposure of the pool of sewage over the sand filter bed than to any other cause, so far as the writers can determine.

The sewage, after passing through the filter bed, still contained considerable black material and was generally of a comparatively foul nature. Its discharge was into an existing 4 inch agricultural drain tile, and in spite of the fact that the effluent is unsatisfactory to the designers of the plant, it has run away in this 4 inch drain tile without causing stoppage or any objectionable results, which have as yet become apparent.

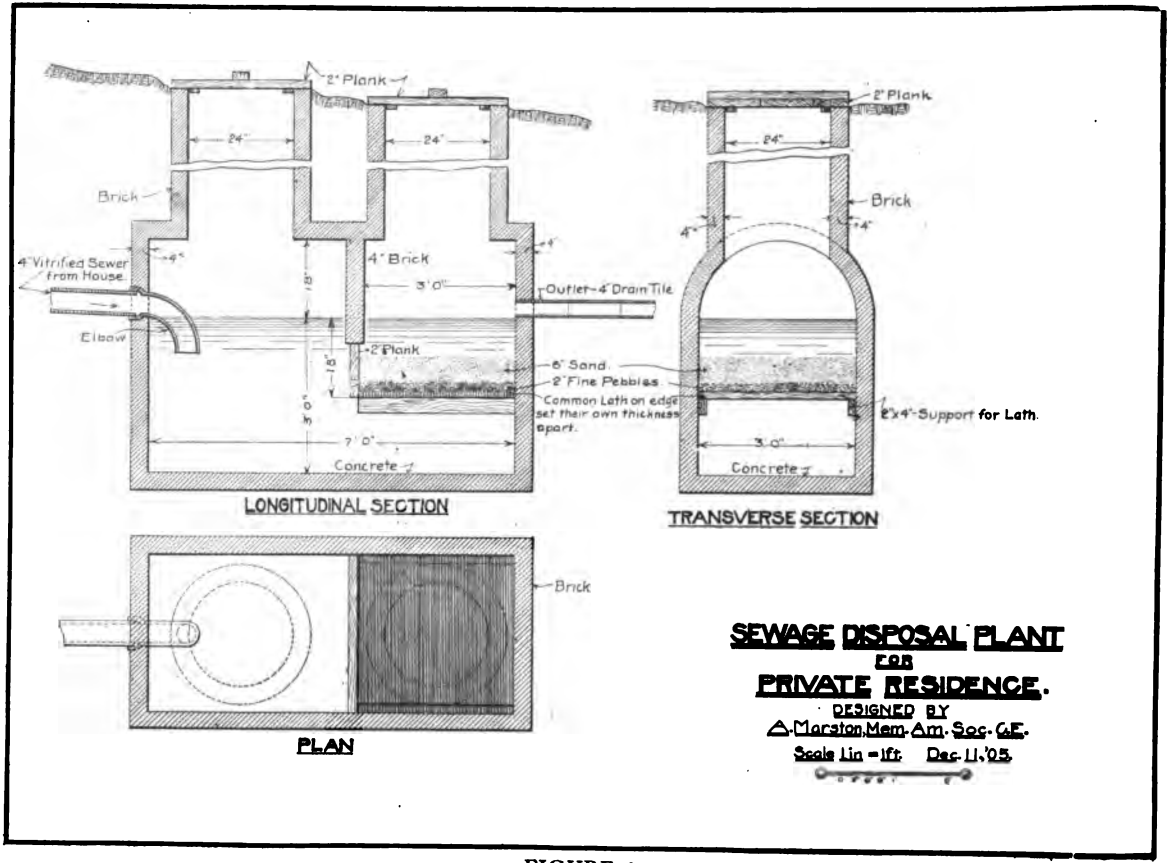

2" Plank
24"
24"
Brick
4"
4" Vitrified Sewer from House
Elbow
18"
4" Brick
3'0"
Outlet-4" Drain Tile
2" Plank
18"
3'0"
6" Sand
2" Fine Pebbles
Common Lath on edge set their own thickness apart.
7'0"
Concrete
LONGITUDINAL SECTION
2" Plank
24"
Brick
4"
4"
2"x4"-Support for Lath.
3'0"
Concrete
TRANSVERSE SECTION
Brick
PLAN
SEWAGE DISPOSAL PLANT
FOR
PRIVATE RESIDENCE.
DESIGNED BY
A. Marston, Mem. Am. Soc. C.E.
Scale 1in = 1ft.
Dec. 11, '05.

FIGURE 3

EXPLANATION OF FIGURE 3.

Figure 3 shows a form of septic tank which was designed after experience with plants No. 1 and 2, to obviate some of the difficulties encountered therewith. To prevent, as far as possible, the outflow of sediment, an upward sand filter is provided, as shown in Figure 3. In practice, this filter clogged, in some instances, at the lath grating which supported the pebbles, and the wood strips making this grating should be spaced at least 1 inch apart in the clear instead of as shown in the figure. Considerable fine, black material came out of the tank in spite of the sand filter.

Experiments are still in progress with this type of tank, and in some instances it has given as much satisfaction as could be expected of any tank without filters.

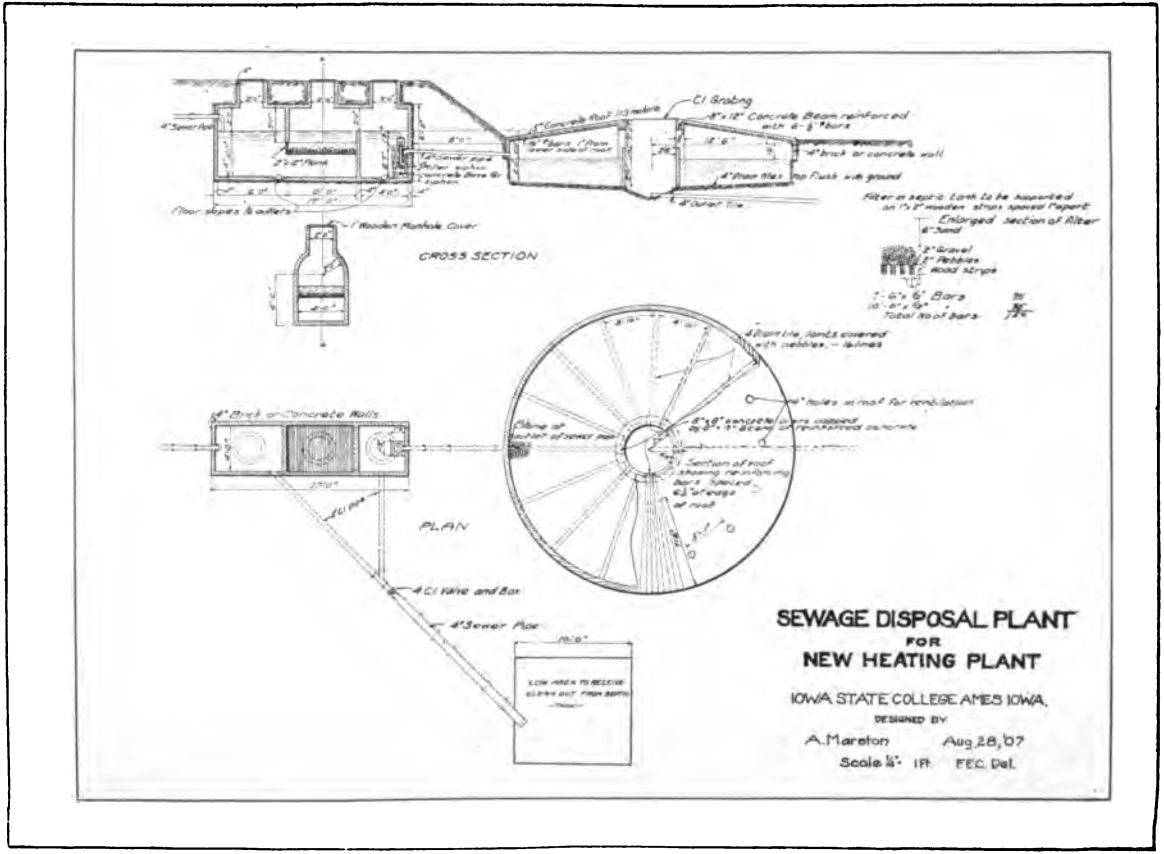

C I Grating
5" Concrete Roof 1:3 mixture
8"x 12" Concrete Beam reinforced with 6-½" bars
12'6"
4" brick or concrete wall
4" Drain tiles
Top Flush with ground
4" Outlet Tile
4" Sewer Pipe
2"x 12" Plank
4" Sewer pipe
Miller siphon
Concrete Base for siphon
6'0"
8'0"
4'0"
Floor slopes to outlets
1" Wooden Manhole Cover
CROSS SECTION
Filter in septic tank to be supported on 1"x 2" wooden strips spaced 1" apart
Enlarged section of filter
6" Sand
2" Gravel
2" Pebbles
Wood strips
Bars
Total No of Bars
4 Drain tile, joints covered with pebbles, — 16 lines
4" holes in roof for ventilation
Plane of outlet of sewer pipe
Section of roof showing reinforcing bars spaced 6½" at edge of roof
4" Brick or Concrete Walls
PLAN
4 C I Valve and Box
4" Sewer Pipe
LOW AREA TO RECEIVE CLEAN OUT FROM BERTH
SEWAGE DISPOSAL PLANT
FOR
NEW HEATING PLANT
IOWA STATE COLLEGE AMES IOWA.
DESIGNED BY
A. Marston
Aug 28, '07
Scale ¼" = 1 Ft.
F.E.C. Del.

FIGURE 3½

EXPLANATION OF FIGURE 3½.

Figure 3½ shows the details of a private sewage disposal plant, which, with temporary omission of cover for filter bed, was built in 1907 for the Iowa State College Central Heating Plant, and which constitutes experimental plant No. 3. The tank is built along the same lines as the one shown in Figure 3 for a dwelling, but is made larger because of the greater volume of sewage from the heating plant. A dosing chamber is added containing a 3 inch siphon, as in plants Nos. 1 and 2, and a circular sand filter 25 ft. in diameter receives the effluent from the tank.

This plant has been in satisfactory operation ever since its construction, but it receives more constant and intelligent care from the employees of the heating plant than could reasonably be expected from the ordinary private house-owner.

EXPERIENCE WITH EXPERIMENTAL PLANT NO. 3.

In view of the experience had with plants Nos. 1 and 2, which was corroborated by observation of another plant built by a private house-owner, of the same design used for plant No. 2, it was felt that a different design would be necessary for a plant which could be recommended for general private use.

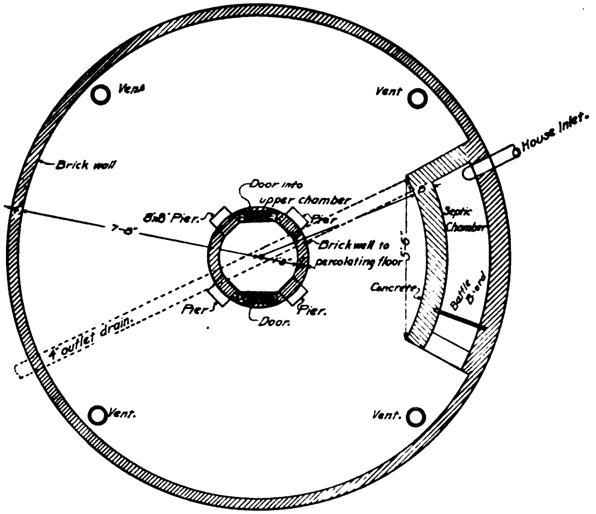
Many requests for plans for private plants were constantly coming in. In view of the unsatisfactory experience with the filters, it was felt that a design for a tank alone might be desirable. The design shown in Figure 3 was accordingly prepared. In this design an attempt was made to secure a comparatively uniform and very gentle outflow by compelling the sewage to pass through an upward sand filter. This filter was supported upon fine pebbles which were in turn carried by a grating of lath on edge.

The results were considerably better than with previous designs of sewage tanks, but more sediment was carried out than had been hoped. In the course of time, there was considerable clogging at the lath grating. These facts were observed on an experimental wooden tank, built according to the same general plan. Experiments are still continuing with this type of tank, using 1 inch clear spacing of the grating bars. It is probable that where it is in use it would occasionally be necessary to remove and replace the sand filter.

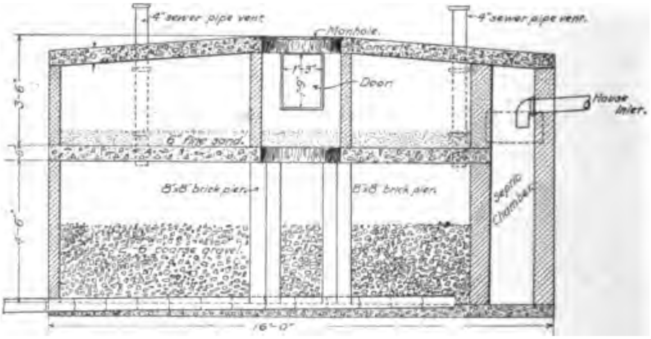
No tank alone will give sufficient purification, and some means of filtration should be provided, unless discharge can be made direct into a considerable stream of water.

In 1907 it became necessary to construct a plant for the college Heating Plant, and the design shown in Figure 3½ was adopted. In construction the cover for the filler bed was temporarily omitted. This plant is larger than would be required for a single family. In operation, it is reported quite satisfactory by the men in charge of the plant. No objectionable odor is noticed from it, although it is quite close to the building. The filter bed is often found flooded, owing, it is thought, to backwater covering the outlet. Flooding of the filter should not occur with free outlet of sewage and with regular care of the sand fillter. Such care would consist of loosening up the surface every month or so during the summer, and occasionally removing some dirty material.

This plant receives more intelligent care than could ordinarily be expected for the private sewage disposal plant. Hence, we do not feel justified in recommending this type of plant for the ordinary private house. We should expect that it would be neglected, that the sand bed would be constantly flooded with sewage, and that the siphon would fail to operate as intended.



PLAN.



SECTION.

FIGURE 4

EXPLANATION OF FIGURE 4.

Figure 4 shows the most satisfactory private sewage disposal plant among those experimented with. It was constructed in 1905. It has operated for three years without any expense or care. It seems to be larger than would be needed for a single family, and would be costly to build.

EXPERIENCE WITH EXPERIMENTAL PLANT NO. 4.

Experimentation was begun in 1905 with the private sewage disposal plant shown in Figure 4. It was constructed on the principle of the sprinkling filter. This plant was built at the house of the college custodian, Iowa State College, Ames, Iowa. In constructing it the workmen put unscreened sand in the bottom filter instead of screened pebbles, as intended, and after a lapse of about one year the sand was removed and replaced with pebbles as at first intended. At this time, the cesspool tank was cleaned out. Otherwise, the plant has not needed any work or expense since its first construction. It has received the sewage of a family of six or eight people, and while not giving a perfectly clear effluent, has purified the sewage to such an extent that it has been discharged on a hillside below the plant for four years without creating a nuisance. At the present time, there is, perhaps, a wheelbarrow load of deposit of the color of lampblack in the little ditch forming the outlet, but this does not seem to cause offensive odor. So far, no especial arrangement has been made to supply a circulation of air throughout the plant, but it is believed that the work of the plant could be materially improved by providing for such air circulation.

As shown in Figure 4, the plant consists, first, of a cesspool tank in which the sewage is received from the house. Second, a sand filter 6 inches thick receives the outflow from the tank. This filter is supported by a perforated reinforced concrete floor, the perforations being of conical shape and filled with pebbles fine enough to hold up the sand. Third, there is a sprinkling filter 2 ft. 6 inches deep at the bottom, composed of pebbles having the sand all screened out.

In the operation of the plant, the sewage covers the upper filter to a depth of a few inches, and continually drops through the upper filter upon the surface of the sprinkling filter below.

TESTS OF EFFICIENCY OF CESS POOLS AND OF EXPERIMENTAL PRIVATE SEWAGE DISPOSAL PLANTS

Tests of efficiency of the plants which have been described were made by Messrs. J. B. Neely and S. R. Green in connection with their thesis investigation in 1907. Messrs. Neely and Green collected the samples. The chemical analyses were made in the laboratory of the Engineering Experiment Station, by Mr. C. E. Ellis. The bacterial analyses were made in the college Bacteriological Department.

In making the tests, it was found very difficult to secure representative samples of the raw sewage. It is believed that this was accomplished by collecting for a time the entire flow of raw sewage in a galvanized iron bushel measure. When this was filled, the contents were stirred and the sample taken. Without some such precaution as this, the sample of raw sewage would have been found to range in quality from pure water to extremely strong sewage.

The bacterial results show in each case the average of 16 separate culutre plates, two of which were poured at each of the following hours: 3 A. M., 6 A. M., 9 A. M., 12 noon, 3 P. M., 6 P. M., 9 P. M. and 12 midnight. The medium used was Agar-Agar, and the counts were made at 48 hours, the standard temperature being 28° C. The results of these chemical and bacterial analyses appear in the table on page 30.

The results for plant No. 2 in this table show the bad effects of continuous flooding of the sand filters. The same is true of the second test in the table, which was made on an experimental wooden tank similar to that in Figure 3 in combination with a sand filter like that in Figure 2.

The third test in the table shows what can be expected from a sewage tank alone, like that in Figure 3. In general, the effluent from such a tank is not sufficiently pure, and requires filtration.

The tests of plant No. 4 give, on the whole, the best results.

The tests of the private cesspools show that their effluents are swarming with bacteria and are extremely foul and impure liquids.

For comparison with the tests of private plants and cess-

CHEMICAL AND BACTERIAL TESTS OF PRIVATE SEWAGE DISPOSAL PLANTS AND CESSPOOLS

DESCRIPTION OF PLANT	Character of Sample	Chlorine	PARTS PER MILLION						Bacteria per CC.
			Albuminoid Ammonia	Free Ammonia	Nitrites	Nitrates	Oxygen Consumed	Loss on Ignition	
Plant No. 2 (See Fig. 2)	Raw Sewage	54	7.5	10.5	....	....	55.0	476	278,000
	Tank Effluent	98	7.5	48.5	....	....	90.1	670	132,000
	Final Effluent	98	42.5	10.0	high	....	71.7	484	367,000
Tank like Fig. 3, Filter like Fig. 2	Raw Sewage	100	16.0	35.0	0	0	147.1	1220	215,000
	Tank Effluent	100	7.0	40.5	....	....	125.1	836	154,000
	Final Effluent	86	2.0	28.5	....	....	86.7	368	58,000
Tank in Fig. 3	Raw Sewage	100	35.0	113.5	....	....	140.0	2056	172,000
	Tank Effluent	100	3.5	92.5	....	....	54.5	410	92,000
Plant No. 4 (See Fig. 4)	Raw Sewage	78	3.0	32.5	....	....	58.7	330	198,000
	Tank Effluent	72	1.5	36.0	....	....	47.5	240	126,000
	Final Effluent	74	1.0	19.0	high	1.3	20.5	170	24,000
Private Cesspool No. 1	Effluent	80	15.0	57.5	....	....	71.7	664	1,200,000
Private Cesspool No. 2	Effluent	132	11.0	117.0	....	....	63.4	450	930,000
Ames City Plant—Septic Tank anl Sand Filters	Raw Sewage	61	7.5	20.0	0	0	96.0	230	470,000
	Tank Effluent	60	5.0	21.0	0	0	99.6	226	845,000
	Filter Effluent	80	0.6	0.1	....	4.8	20.2	250	900

pools, an analysis of a city plant in good working order is given in the table on page 30. Comparing it with the other tests, we find a much higher degree of purification in the city plant. In the private plants the nitrates show that no nitrification was effected, except in the case of plant No. 4.

SUMMARY OF CONCLUSIONS FROM EXPERIENCE WITH EXPERIMENTAL PRIVATE SEWAGE DISPOSAL PLANTS.

Our experience with the four types of private disposal plants already described leads us to the following general conclusions concerning sewage disposal plants for private houses:

1. *Size of tank.* The size of sewage tank for the private house should be much larger per person than in the case of city plants, on account of the extremely fresh character of the sewage in the former case, and its widely fluctuating quality and rate of flow.

2. *Prevention of disturbing effect of inflow into the tank.* It is especially necessary, in the case of private sewage disposal plants, that there should be provided some device for completely destroying the entering current of sewage, which, otherwise, stirs up the entire contents of the tank, owing to its small dimensions, and disturbs the purifying agencies.

3. *Prevention of disturbing effect of outflow from the tank.* Similarly, owing to the small dimensions of the private tank, it is necessary to use some special device for taking away the outflow without creating perceptible currents, as such currents will carry away sediment which will clog the filter beds.

4. *Use of mechanical apparatus not allowable in private plants.* We find that the use of any mechanical appliances, such as siphons or other automatic devices, is not permissable in the case of private plants, as all such devices require more or less attention to insure their proper working, and are almost certain not to receive such attention from the average householder. When out of order, they are fatal to the satisfactory operation of the plant.

5. *Area of filters.* Private plants require several times the area of filter per person needed for city plants because of the lack of corresponding care, and because of the extra amount of sediment carried from the tank in spite of all possible precautions.

6. *Care of plant.* No matter what the design, the householder must give some care to the sewage disposal plant. With the design which will be recommended, it is believed that such care will be confined to overhauling and cleaning once per year.

7. *Purity of effluent.* So far, it has proven impracticable to secure an effluent equal to the high degree of purity obtained from the city intermittent sand filter plants, but the effluent from the plant recommended should be sufficiently purified to prevent further offensive putrefaction.

DESIGN RECOMMENDED BY THE IOWA STATE COLLEGE ENGINEERING EXPERIMENT STATION FOR SEWAGE DISPOSAL PLANTS FOR PRIVATE HOUSES.

As the result of our experimental work extending over a period of five years, we have prepared the design shown in Figure 5 as embodying our present ideas. We recommend this design, but with the caution that no actual plants of the exact details shown have as yet been constructed. We hope and expect to be able to make advantageous modifications of this design from the results of our further work. We ask detailed information from those who build plants according to this design, stating the cost of construction and their experience with the working of the plants. We will warmly welcome intelligent criticism. Hence we ask for full records of unfavorable as well as favorable experience with the plants.

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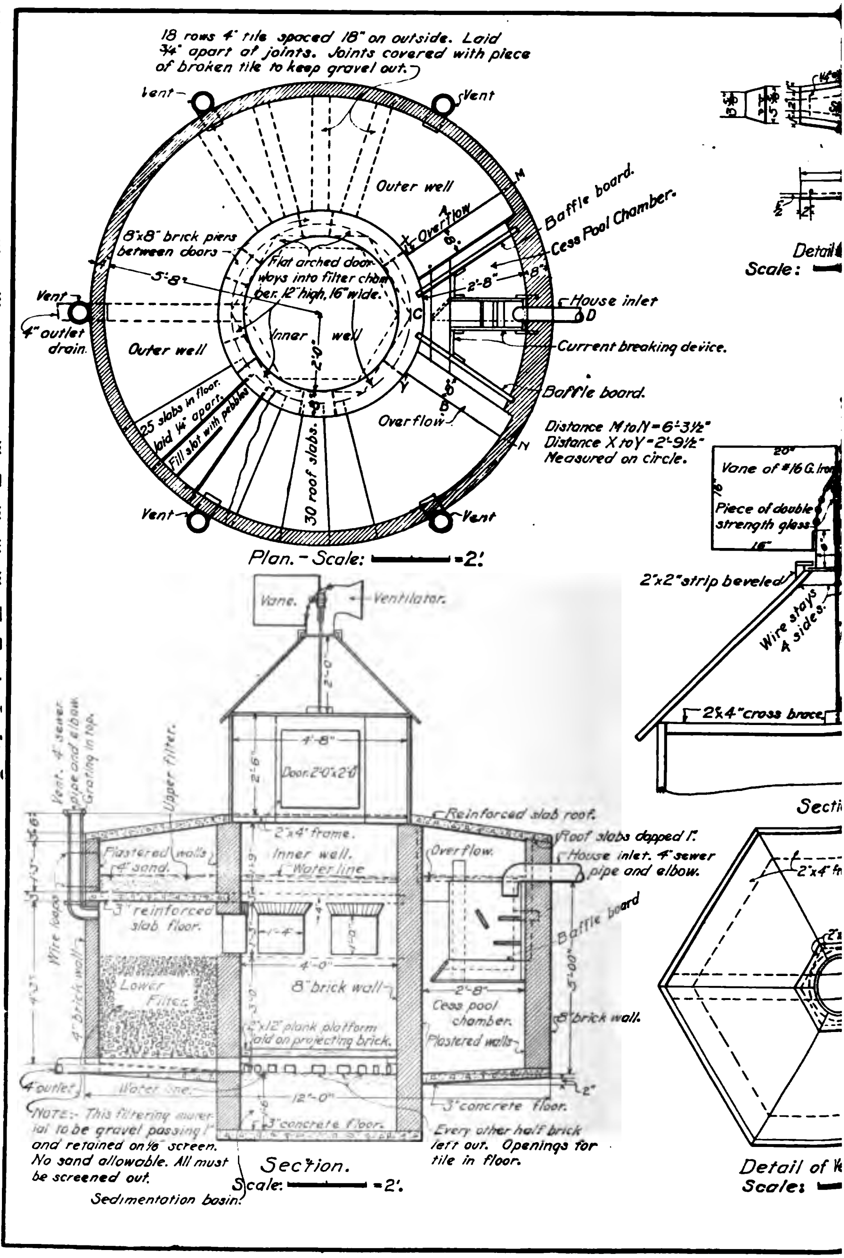
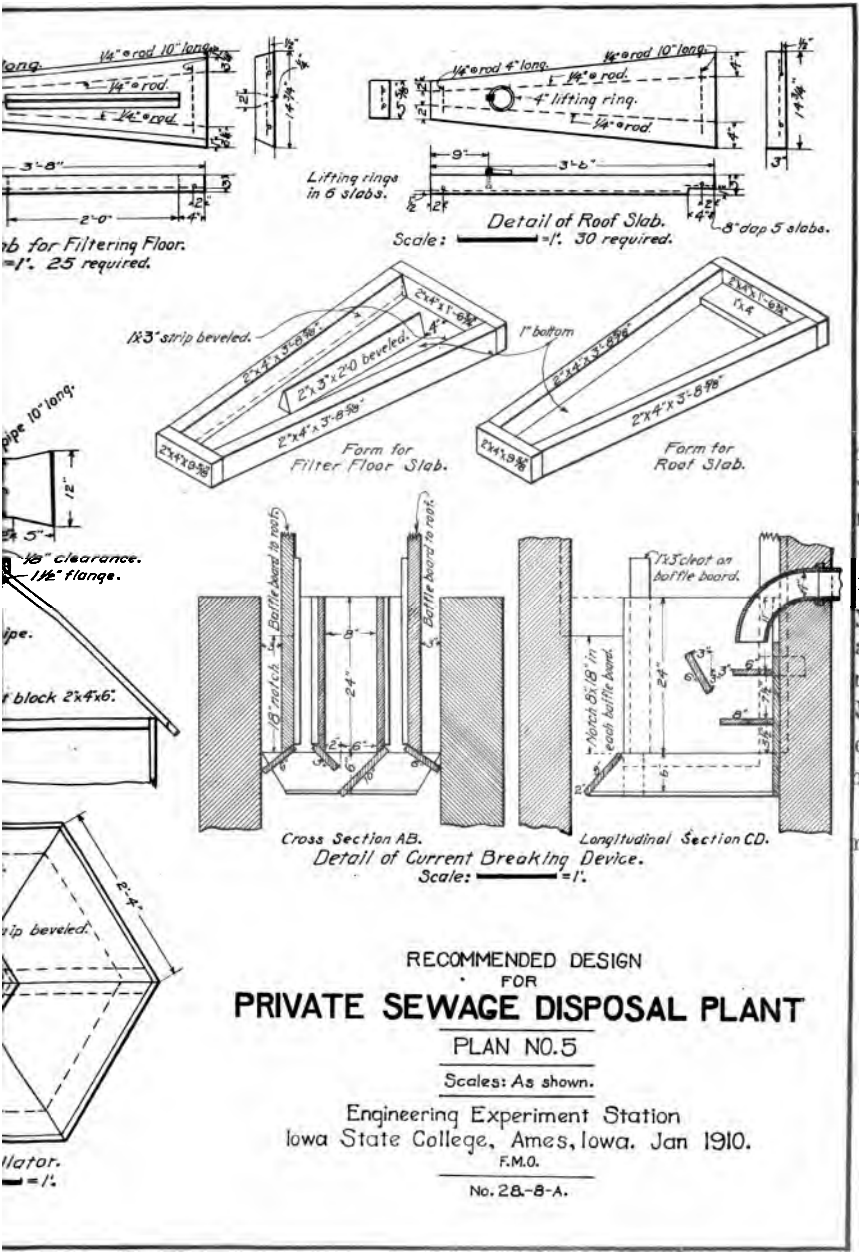

18 rows 4" tile spaced 18" on outside. Laid 3/4" apart at joints. Joints covered with piece of broken tile to keep gravel out.
Vent
Vent
Outer well
M
A
Overflow
8"
Baffle board.
Cess Pool Chamber.
8"x8" brick piers between doors
5'-8"
Flat arched doorways into filter chamber. 12" high, 16" wide.
2'-8"
8"
C
House inlet
D
Vent
4" outlet drain.
Outer well
Inner well
2'-0"
Current breaking device.
X
Y
25 slabs in floor. laid 1/4" apart.
Fill slot with pebbles
Baffle board.
8"
B
Overflow
30 roof slabs.
N
Distance M to N = 6'-3 1/2"
Distance X to Y = 2'-9 1/2"
Measured on circle.
Vent
Vent
Plan. - Scale: = 2'.
Vane.
Ventilator.
2'-0"
4'-8"
2'-6"
Door 2'0"x2'0"
Vent. 4" sewer pipe and elbow. Grating in top.
Upper filter.
Reinforced slab roof.
Roof slabs dapped 1".
House inlet. 4" sewer pipe and elbow.
2"x4" frame.
Plastered walls
4" sand.
Inner well.
Water line
Overflow
3'-6"
1'-3"
3"
6"
1'-3"
1'-4"
1'-0"
4"
3" reinforced slab floor.
Baffle board
Wire loops
4" brick wall
4'-3"
Lower Filter
4'-0"
8" brick wall
3'-0"
2'-8"
Cess pool chamber.
5'-0"
8" brick wall.
2"x12" plank platform laid on projecting brick.
Plastered walls
4" outlet
Water line.
12'-0"
1'-6"
2"
3" concrete floor.
3" concrete floor.
NOTE:- This filtering material to be gravel passing 1" and retained on 1/8" screen. No sand allowable. All must be screened out.
Section.
Scale: = 2'.
Sedimentation basin.
Every other half brick left out. Openings for tile in floor.
Detail
Scale:
20"
Vane of #16 G. Iro
16"
Piece of double strength glass
16"
2"x2" strip beveled
Wire stays 4 sides.
2"x4" cross brace
Secti
2"x4" fr
Detail of V
Scale:

FI



RECOMMENDED DESIGN
FOR
PRIVATE SEWAGE DISPOSAL PLANT
PLAN NO. 5
Scales: As shown.
Engineering Experiment Station
Iowa State College, Ames, Iowa. Jan 1910.
F.M.O.
No. 28-8-A.

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DESCRIPTION OF FIGURE 5.

Figure 5 shows the design of private sewage disposal plant recommended by the Iowa State College Engineering Experiment Station as embodying the best ideas obtained by five years of experimental work on such plants.

The plant consists of a cesspool chamber, a two-story filter, (the upper filter being sand and the lower pebbles) and, finally, of a sedimentation basin. The whole is enclosed in a brick masonry well, 12 ft. in diameter by 7 ft. deep, with a wooden ventilator reaching about 5 ft. above the surface of the ground. No plant of the exact description of the plant shown has as yet been built and tried out. Farmers should be able to build the plant for about $100, in addition to their own work.

Further results of our experimental work may lead to improvements in this design.

DETAILED DESCRIPTION OF DESIGN RECOMMENDED.

The design recommended is shown in Figure 5. It consists of an *outer well*, of brick, 12 ft. in diameter outside, and an *inner well*, also of brick, 4 ft. in diameter inside. A *sedimentation basin* is added at the bottom of the inner well, and a *ventilator* at its top. One sixth of the outer well is partitioned off to serve as a *cesspool chamber*.

Outer and Inner Wells. The brick wall of the outer well is 5 ft. 10 in. in depth, and is 4 inches in thickness, except for 1-6 of the circumference which is 8 inches thick. The brick wall of the inner well is 7 ft. 11 in. deep, its top being 5 inches higher and its bottom 1 ft. 8 in. lower than the corresponding parts of the outside wall. The wall of the inner well is 8 inches thick. These brick walls and all other brick work about the plant must be laid up in cement mortar, composed of 1 part Portland cement to 3 parts of good, clean, coarse sand. No lime whatever must be permitted about the plant. The bottom of each well is composed of 3 in. of concrete, which must be mixed in the proportion of 1 part cement, to 2 parts of sand, and 3 parts of broken stone or of pebbles not containing sand. It can be made, if desired, of 1 part of cement, to 4 parts of clean gravel containing a considerable proportion of pebbles.

Sedimentation Basin and Outlet Pipe. The inner well extends 1 ft. 6 in. deeper than the outer well, and forms a sedimentation basin.

The outlet from this sedimentation basin, and from the entire sewage disposal plant, is provided by a 4 inch tile drain, laid with its inner end 1 ft. 6 in. above the bottom of the inner well. This outlet should be extended to the place of final discharge, into some ditch or on a hillside. Just above this outlet tile, two 2 in. x 12 in. plank must be laid across the inner well, supported on projecting brick at each end, so as to form a platform above the water in the sedimentation basin, on which to stand when examining the plant or working with it from the inner well.

Lower Filter. The floor of the outer well has a slope of 2 inches toward the inner well. Radiating out from the inner well are laid 18 straight lines of 4 inch tile, spaced 18 inches at the outside. The pieces of tile must be laid ¾ inch apart at the joints, and the upper halves of the joints must be covered with broken tile to prevent the entrance of filter pebbles into the tile. The outlets for these tile are provided by omitting every other half brick from the wall of the inner well. The tile serve partly to afford a supply of air to the under part of the lower filter, and partly as outlets for the filtered sewage.

The lower filter occupies the lowest 3 ft. of the outer well, and must be composed of fine broken stone with no dust, or of pebbles with all the sand screened out. In either case, the range of size must be from not less than 1-6 inch up to not more than 1 inch in any dimension. In case pebbles are used, a screen having 1-6 inch mesh can be used to screen out the sand, and all the coarse pebbles in the pile can be broken up with a sledge or hammer. About 8 cubic yards of this material will be required for the lower filter.

Space between lower and upper filters. Beginning 1 inch above the upper surface of this lower filter, six rectangular openings, each 12 inches high by 16 inches wide, are provided in the wall of the inner well. This is to permit free access to the lower filter to examine it and introduce or take out filtering material. They also permit the free passage of air from the ventilator through the space between the lower and upper filters and out through the five vents placed around the outside of the outer well, as shown in Figure 5.

Vents. Each of these vents consists of a 2 ft. length of 4 inch vitrified sewer pipe, attached to a 4 inch vitrified elbow, passing through the outer wall in the position shown in Figure 5. The 4 inch sewer pipe are each attached to the outer wall by 2 wire loops.

The upper ends of the vents must be stopped by pieces of wire screen, to prevent objects being thrown down the vents, while permitting the free escape of air.

Floor of upper filter. The floor of the upper filter consists of reinforced concrete slabs, of the dimensions and other details shown in the separate plan in Figure 5. These slabs are 3 inches thick, and each is reinforced by 2 iron or steel rods 1/4 inch round by 3 ft. 8 in. long, placed 1/2 inch from the bottom of the slab, together with 2 small cross rods, as shown in Figure 5. The edges of each slab are beveled, as shown in Figure 5, and there is a beveled slot 2 ft. long, also as shown in Figure 5. The object of these slots and beveled edges is to permit the passage of the sewage from the upper filter. The beveled slots are to be filled with small pebbles, sufficiently fine to hold up the sand forming the upper filter. These pebbles should not be more than 1/8 to 1/2 inch in diameter, and in the upper portion should not be larger than 1/8 inch in diameter.

Water must be turned upon the surface of the sand in the upper filter during construction, to test it and make sure that the sand does not wash out through any of the slots, and should this occur, finer pebbles must be used and the sand then replaced. This test must be continued until it is certain that the upper filter is perfectly supported, so as not to be in danger of washing out.

The floor slabs are to be constructed in molds, plans for which are shown in Figure 5. The slabs must be made of 1 part Portland cement, to 2 parts sand, and 3 parts broken stone or small pebbles with the sand all screened out; or, if preferred, they may be made of Portland cement and gravel, in the proportion of 1 part of cement, to 4 parts of gravel containing a considerable proportion of pebbles. By molding these at the surface of the ground, we avoid the necessity of wooden forms or centering to hold the floor during construction.

Upper filter. The upper filter consists of 4 inches of coarse, clean sand. About one good load is required for the filter. The walls of the upper filter must be plastered with 1 to 2 Portland cement mortar, as shown in Figure 5.

After the plant has been in use for some time, the sewage will entirely cover the surface of this upper filter and finally reach a considerable depth. Under these circumstances, the sewage will filter through the upper filter and pass down through the slots in the supporting floor and fall upon the surface of the lower filter in the form of separate drops. These drops will be exposed to the air as they fall and take up oxygen from it. In this way, a supply of oxygen for the lower filter will be maintained.

This upper filter will, in course of time, become so clogged with the sewage that the sewage will reach the height of the outer wall. When this occurs, a sufficient amount of the roof should be lifted off to permit the sand of the upper filter to be taken out and replaced with new sand. Only about one load will be required. It is not certain how often this removal will need to be made, but it will probably be not oftener than once in two years.

Roof. The roof of the outer wall is also made of concrete slabs of the same dimensions as those of the floor of the upper filter, which are to be made in molds in the same way. Plans for the roof slabs and molds are shown in Figure 5. Like the slabs in the filter floor, each must be made in the proportion of 1 part Portland cement, to 2 parts sand, to 3 parts of fine broken stone or pebbles free from all sand; or, if preferred, they may be made from clean gravel, in the proportion of 1 part of cement, to 4 parts of good gravel containing a considerable proportion of pebbles. The roof slabs differ from the filter floor slabs in that they are not beveled, and that they are notched at one end so as to be held in place by the outer wall. They are reinforced by ¼ inch round rods just as in the case of the filter floor slabs.

All the roof slabs are removable, and one in every six is to be provided with a lifting ring at the small end. This lifting ring can readily be made of twisted wire.

Cesspool chamber. The sewage entering the tank is first received in a cesspool chamber, as shown in Figure 5. From outside to outside of walls, this cesspool chamber occupies exactly 1-6 of the circumference of the entire disposal plant. The inner well forms one wall of the cesspool. The radial walls are each 8 inches thick, made of brick. The outside wall of the cesspool chamber is 8 inches thick instead of 4 inches, as in the case of the remainder of the outside wall. The interior walls of the cesspool chamber must be plastered with a 1 to 2 Portland cement mortar.

The sewage enters the cesspool chamber by a 4 inch vitrified pipe sewer from the house, as shown in figure 5. This pipe ends in a 4 inch elbow which serves as a trap to prevent any escape of gas from the cesspool chamber into the house sewer.

The entering flow is broken up and kept from stirring up the sediment by the wooden device shown in Figure 5. It is important that this device should be constructed exactly as shown in the figure, for without it the entering current will stir up the sediment in the cesspool chamber, disturb the bacterial agencies of purification, and carry the sediment out upon the sand filter.

The sewage from the cesspool chamber overflows along the entire length of each side wall upon the surface of the upper sand filter. To reach the overflows, the sewage must pass behind baffle-boards, which extend from the roof line down to a point 2 ft. below the entering line of the house sewage, as shown in Figure 5. These baffle-boards must be constructed of 2 in. x 12 in. planking.

After the plant has been in operation some time, the sewage will cover the entire surface of the upper filter, as already described, so that the space between the sand of the upper filter and the under side of the roof will constitute an extension of the cesspool chamber. When this occurs, the effect of any sudden inrush of water will simply be to raise very slightly the level of the sewage. The velocity of outflow from the cesspool chamber will the be very slight. Under these conditions, we anticipate a maximum retention of solids in the cesspool chamber.

Ventilator. The ventilator is to be constructed as shown in Figure 5. Except for the 2 in. x 4 in. frames, it is to be made of 1 inch boards. It is hexagonal (six-sided) in shape, just covering the 4 ft. inner brick well. On one side should be placed a 2 ft. x 2 ft. door with 2 in. x 4 in. frame. This door should be hinged and provided with hasp, staple and padlock. It serves to admit entrance to the sewage disposal plant whenever desired.

At the vertex of the pyramidal roof is placed a galvanized iron ventilator, which can be made by any competent tinner. The detailed view of this ventilator is shown in Figure 5 to a large scale. It is 12 inches in diameter at the point where the air enters and 9 inches at the elbow. It is kept pointed in the direction of the wind by a vane composed of a sheet of galvanized iron, as shown in Figure 5. To the interior of the elbow is riveted a socket composed of a piece of 1 inch pipe, 10 inches long. In the upper end of this a piece of double strength glass is fastened by Portland cement, so as to afford an anti-friction bearing on which the ventilator is to turn. It is important that this piece of glass be used, as it is otherwise possible that the ventilator might stick and not turn with the wind. The socket is upheld by a piece of ½ inch pipe, stayed at the vertex of the roof by iron wires, and supported by a 2 in. x 4 in. foot block, resting on a 2 in. x 4 in. cross brace, all as shown in Figure 5. At the lower part of the galvanized elbow a 1½ inch galvanized iron flange must be formed, as shown in Figure 5, fitting into a wooden slot, also as shown in the figure, to prevent the escape of air.

The object of this ventilator is to insure a current of air through the disposal plant. The vane keeps the ventilator turned toward the wind, whose force will therefore drive a current of air down into the interior of the inner well. After aerating the falling sewage and the drain tile pipes in the under part of the lower filter, this current of air will escape through the five 4 inch vents spaced along the outside wall. This aeration of the sewage in the lower filter is necessary to the successful working of the plant.

BILL OF MATERIAL FOR DESIGN RECOMMENDED

The following is an approximate bill of material for the design of private sewage disposal plant recommended, as shown in Figure 5, and as just described in detail.

We have also added approximate prices, but these will vary in different localities, and each person should ascertain the local prices for himself.

BILL OF MATERIAL

3000	common brick @ $10.00	$30.00
12	bbl. best Portland cement @ $2.00	24.00
110	rods, ¼ in. round x 3 ft. 8 in.	3.00
55	rods, ¼ in. round x 0 ft. 10 in.	
55	rods, ¼ in. round x 0 ft. 4 in.	
3	pcs. 2 in. x 12 in. x 16 ft. common lumber	9.00
8	pcs. 2 in. x 4 in. x 12 ft. common lumber	
10	pcs. 1 in. x 10 in. x 16 ft. common lumber	
1	pc. 1 in. x 12 in. x 16 ft. common lumber	
1	pr. hinges	.50
1	hasp and staple	
1	padlock	
1	pc. galvanized screen, 12 in. x 18 in.	
1	galvanized iron ventilator	3.00
6	2 ft. lengths 4 in. vitrified sewer pipe	4.00
6	4 in. vitrified sewer pipe elbows	
70	ft. 4 in. drain tile	
6	cu. yd. sand	
11	cu. yd. broken stone or pebbles with all sand screened out, 1/6 inch to 1 inch	
		$73.50

Plus cost of sand and pebbles.

SUGGESTIONS FOR CONSTRUCTING PRIVATE SEWAGE DISPOSAL PLANT RECOMMENDED

Two cases present themselves in connection with the construction of a private sewage disposal plant:

First. Where the householder is prepared to do his own hauling and assist personally in the construction of the plant; and

Second, where he must hire everything done.

In the *first case*, where the householder is prepared to do his own hauling and assist personally in the work, the method of procedure of constructing a plant might well be about as follows:

1. Purchase and haul all materials, including galvanized ventilator.

2. Construct the reinforced concrete roof slabs and upper filter floor slabs, using the molds shown in figure 5, using 1 part of cement, to 2 of sand, and 3 of pebbles or broken stone. In the bill of materials enough lumber is provided to construct five molds for roof slabs and five for upper filter floor slabs. The concrete must be thoroughly mixed, and put into the molds in a condition so wet as to flow freely. The reinforcing rods must be put in place the last thing, and must be imbedded just deep enough to be completely covered by the cement so as to be protected from rusting. Of course, the slabs are constructed upside down. They must be left in the molds one day before being taken out, and should then be wet down and covered so as to prevent injury from drying out. To make enough slabs with this number of molds requires six days, but only a part of the time each day need be given to the work. It may be considered preferable to make a larger number of molds, which can be done for a slight added expense for material.

3. While the roof slabs are under construction, make the excavation for the filter plant. This should be about 13 ft. in diameter, so as to leave ample room for the masonry. It should not be made until all the material is on hand nor until it is certain that a mason can be ready to build the brick walls as soon as the excavation is completed.

4. A skilled mason should have been engaged to assist in the construction of the plant, and he should begin work, with the assistance of the householder, as soon as the materials are on hand, the roof and filter floor slabs completed and the excavation made. With the assistance of the householder, this mason then proceeds to build the plant.

5. The outlet tile must be laid and the concrete floor first constructed, using 1 part of cement to 2 parts of sand and 3 parts pebbles or broken stone.

6. The inner and outer brick walls must then be built up to a level ready to receive the floor of the upper filter.

Use 1 part cement to 3 parts sand for the mortar for all brick work.

Note: Be careful to keep the true circular shape so that the reinforced concrete slabs may fit. Be sure to omit every other half brick at lower filter floor level, and to leave projecting brick to carry two plank over sedimentation basin. Be sure to put in place the wooden baffle-boards and current breaking device in the cess-pool chamber. Be sure to construct rectangular openings from the inner well to the space between the lower and upper filters, and to set in place in the outer brick wall the vitrified elbows for the five air vents.

7. The pebbles for the lower filter must be put in place.

8. The reinforced concrete slabs forming the floor of the upper filter must be put in place.

9. The construction of the upper part of the brick walls must be completed.

Note: Be sure to make the junction of these walls with the floor of the upper filter water-tight, and to plaster with 1 to 2 cement mortar the brick walls of the upper filter and of the cesspool chamber, as indicated in Figure 5.

Set in place in the brickwork of the cesspool chamber the inlet sewer, and complete the wooden device for breaking up the current, as shown in Figure 5. Note that the baffle-boards in the cesspool chamber extend clear up to the roof line, and must be cut to the right slope.

10. Construct the six-sided wooden ventilator as shown in Figure 5. The galvanized iron ventilator should have been se-

cured from a good tinner in advance of beginning the construction.

Note: The wooden ventilator must be neatly painted on final completion of the work.

11. Put in place the sand of the upper filter. Be sure to fill the slots in the floor of this filter with very small pebbles and take especial pains to see that the upper portion of the slots is filled with pebbles small enough to hold up the sand of the upper filter. When the sand of the upper filter is in place, pour water all over it to make sure that the sand will not wash down through the pebbles in the slots. It is very important that this test be made thoroughly, and any defects remedied before the sewage is turned in.

12. Complete the five 4 in. air vents around the outside of the tank.

13. Put the roof slabs in place.

14. Thoroughly clean out the inner well and all other portions of the disposal plant. Place the plank platform over the sedimentation basin.

15. Complete the outlet from the disposal plant to the point of final discharge, and the inlet sewer from the house to the disposal plant.

In the *second case*, where the householder must hire everything done, he may proceed as in the first case, hiring teams and men to do the hauling and excavating and other labor which he cannot perform himself. A second method would be to award a contract to a skilled workmen to build the entire plant, the householder furnishing the materials. A third method would be to award a contract for the entire plant, the builder to furnish all the materials. The second of the above methods will often be found preferable, but local conditions must determine the method used.

INSTRUCTIONS FOR OPERATING THE PRIVATE SEWAGE DISPOSAL PLANT

The householder is advised to examine his sewage disposal plant from time to time making sure that it is operating satisfactorily.

About once a year, most of the contents of the cess-pool chamber should be pumped out. The sediment found therein can be plowed under in any garden or disposed of in the same way as other manure. It will be found to have decomposed to such an extent as not to be very offensive to handle. If a pump is not available, the contents of the cess-pool chamber may be dipped out with a pail. Only the bulk of the sediment need be removed. While it is believed that this should be done once a year, yet a roof slab may be removed and the depth of the sediment tested with a sounding rod. If little or no sediment is found, this cleaning out will not be necessary.

In a similar way, the sedimentation basin at the botom of the inner well should be pumped or dipped out whenever the sediment accumulates to any considerable extent. Unless examination should show the absence of sediment, it is advised that this cleaning out should be done once a year.

As already stated, there is danger that the sand of the upper filter might break down through the supporting pebbles in the slots of the filter floor unless these have been carefully placed. Examination should be made from time to time, especially until the sewage has entirely covered the upper filter to a depth of several inches, to make sure that such breaking through has not occurred. Any breaks which do occur must be remedied at once.

At intervals which cannot be foretold, but which it is considered probable will not be oftener than once in two years, it will be necessary to take out the sand of the upper filter and replace it with clean material. About one load of material will be necessary.

CAUTION CONCERNING PATENTS.

Patents have been issued in the United States and England on the septic process of purifying sewage, and the validity of the U. S. patent, so far as it relates to the general process, has been upheld in the United States Circuit Court of Appeals for the district in which the State of New York is located. It is claimed that these patents expired in November, 1909, but this is denied by the owners of the patents. While the authors of this bulletin believe that the cess-pool chamber forming a part of the sewage disposal plant recommended is, scientifically, simply a cess-pool such as has been constructed from time immemorial, though slightly modified in this case, yet we do not guarantee that the holders of the septic process patent may not claim infringement of their patent by the design shown. Any persons building the private disposal plant recommended, must do so entirely upon their own responsibility.

BLUEPRINT PLANS OF THE SEWAGE DISPOSAL PLANT FOR PRIVATE HOUSES CAN BE OBTAINED FROM THE ENGINEERING EXPERIMENT STATION, AMES, IOWA.

We shall be pleased to send out free of charge, a blueprint plan of the sewage disposal plant described in this bulletin to anyone who will write for the same to A. Marston, Director, Ames, Iowa, and who will make a statement that he proposes to build the plant. Copies of this bulletin can be obtained in the same way.

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